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RELIABILITY OF STRUCTURAL FIRE DESIGN

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ABSTRACT

In general, because of the unpredictable nature of fire and the various uncertainties related, for example, to material properties at elevated temperature, the reliability of structural fire design can be justifiably questioned.

In this project, a typical structural steel design for fire condition is assessed for its reliability. The assessment consists of estimating the probability of failure of structural steel elements exposed to a wide range of fully developed fires. A number of scenarios to account for different passive protection systems and the variability in properties of related parameters are modelled. The main tool of analysis is Monte Carlo simulation using a software named @RISK. The estimated probabilities of failure or reliability indices are measured against acceptable or target values so that definite conclusion with regards to safe or unsafe design can be made. The target probability of failure and the reliability index are also worked out in this project.

The overall results show that applying reliability assessment to structural fire design is of great value in pointing out shortcomings in the design and in enhancing the performance assessment of real structures.

ACKNOWLEDGEMENTS

On completion of the MEFE program and especially this project, it is certainly not a perfunctory act on my part to say "thank you" to all those who have seen me through the whole course.

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Professor David Elms ignited in me an interest in all things related to risk and, as one of my supervisors, instructively questioned my presumptions and provided insights that improved my understanding of the problem.

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RELIABILITY OF STRUCTURAL FIRE DESIGN

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1 INTRODUCTION

1.1 IMPORTANCE OF STRUCTURAL STABILITY

Structural stability of a building during fire constitutes one of the most important aspects of the overall fire safety of the building. In a fire situation, building should not collapse prematurely, if at all, and cause injury to the occupants or firefighters. Structural collapse normally means that building damage is more extensive and hence the economic losses are greater. To better appreciate the importance of designing and constructing building for structural stability during fire, one only needs to look at related statistics. As an example, structural collapse is one of the leading causes of firefighters' deaths in the United States (Dunn 1988). Figure 1 below shows the distribution of firefighter deaths by cause of injury and by nature of injury in the United States in the year 1993 (NFPA 1994).

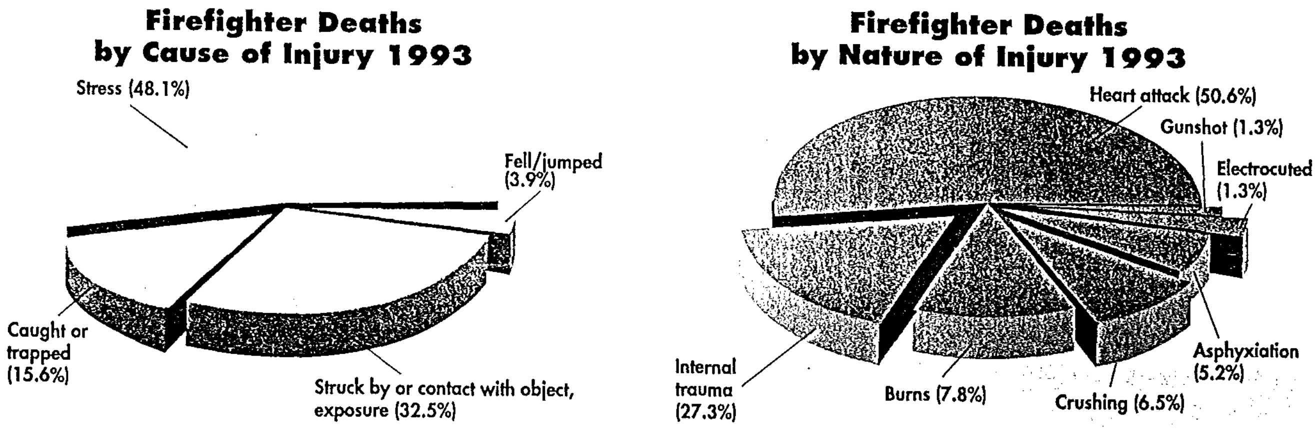


Figure 1 Distribution of Firefighter Deaths by Cause of Injury and by Nature of Injury in the US in 1993 (NFPA 1994)

The figure shows that, in the United States in 1993, "struck by or contact with object" made up 32.5% of the cause of death to firefighters. This, and the other cause "caught or trapped" (15.6%) can be directly attributed to structural collapse of burning buildings. Structural collapse is deemed to have occurred even if a single structural element were to fail under a predefined failure criterion.

1.2 RELIABILITY OF DESIGN FOR STRUCTURAL STABILITY

Designing for structural stability during fire is one of the most difficult tasks faced by engineers (Fitzgerald 1997). This is due mainly to the extremely unpredictable nature of both the fire behaviour and structural responses of the building elements to fire. Design methodologies are based on very empirical studies of fire behaviour and the associated structural responses. The whole analysis contains a lot of approximations and assumptions. In short, design methodologies are always fraught with uncertainties and this raises the important question of how reliable is the structural fire design. .

1.3 OBJECTIVE OF PROJECT

The objective of this project is to illustrate why and how reliability assessment should be applied to structural fire design in order to achieve safe and reliable performance. Reliability assessment will specifically be carried out on typical fire engineering design of steel structure to illustrate the whole process.

1.4 OUTLINE OF METHODOLOGY

To achieve this objective, this project would first broadly preview the present structural fire design methodologies. The preview would then move into the field of quantitative risk assessment and touches on the areas of uncertainty and reliability analyses. The most appropriate quantitative tool would then be utilised in case studies involving real examples of performance of building elements under fire situation. The results of the case studies will form the basis of discussions about what constitutes acceptable probability of failure or reliability, and the possible implications on the codes, standards, or even the field of fire engineering in order to achieve the target probability of failure and reliability index.

2 FIRE RESISTANCE AND FIRE SEVERITY

2.1 OVERVIEW

In fire engineering, and particularly with regards to structural fire design, the terms fire resistance and fire severity are frequently used. In fact any one term cannot be fully described and understood without attempting to do the same for the other. For this reason and because the terms are used extensively in this project, it is considered necessary to devote a section for explaining these terms as they are used in this report.

2.2 FIRE RESISTANCE

As a term that is frequently ascribed to the behaviour of building components under fire condition, fire resistance is a measure of a building component's ability to resist a fire. More specifically, the fire resistance of a building component or assembly is its ability to withstand exposure to fire without loss of load bearing function, or to act as a barrier against spread of fire, or both. It is often quantified as the time for which the element is expected to meet certain criteria when exposed to a standard fire resistance test.

In most national standards the criteria are:

(i) Stability – resistance to structural collapse

(ii) Integrity – resistance to flame penetration

(iii) Insulation – resistance to excessive temperature on the unexposed face

Thus, the measurement of fire resistance has 3 components - stability, integrity and insulation, all of which are expressed in unit of time.

2.2.1 Fire Resistance Rating

New Zealand's NZS 1900 Model Building Bylaw(1) defines fire resistance rating as:

> "Fire Resistance Rating (FRR) means the minimum period of time for which all sides of an element of structure, any of which is subjected to a standard fire, continues to perform its structural function and does not

> permit the spread of fire. Where a period of time is used in conjunction with the abbreviation FRR, it is required that the element of structure referred to shall have a resistance rating of not less than the period stated"

Fire resistance ratings are usually assigned in whole numbers of hours in order to allow easy comparison with fire resistance requirements specified in building codes. For example, a wall that has been shown by test to have a fire resistance of 75 minutes will usually be assigned a fire resistance rating of one hour.

Fire resistance of any building elements depends on many factors, including the severity of the fire, the material, the geometry and support condition of the element, restraint from the surrounding structure and the applied load. Hence, fire resistance is not a material property in the sense that the yield strength is.

2.2.2 Methods of Determining Fire Resistance

With the recent advance in fire science and technology, there are now basically three different ways of determining fire resistance rating of structural members and assemblies. These three approaches are (i) laboratory testing, (ii) empirical correlations and (iii) theoretical calculations. (Lie 1992). They will be very briefly described below.

2.2.2.1 Laboratory Testing

Full scale testing is the most common method of obtaining fire resistance ratings. These tests are done in special test furnaces and should be carried out in accordance with procedures. The most widely used of these procedures are described in the "Standard Methods of Fire Tests of Building Construction and Materials" ASTM E119 (ASTM 1988). This test method is used to evaluate walls, partitions, beams, columns, floor, and roof assemblies. Similar procedures are used for determining the fire resistance of door and window assemblies. In addition to ASTM, other organisations such as the National Fire Protection Association (NFPA), Underwriters Laboratories Inc (UL), and the British Standard Institution (BSI) also publish fire test methods which are virtually identical to those developed by ASTM and are generally considered to be equivalent.

There are three criteria in the standard test method. They concern load-bearing capacity, integrity and insulation. In many cases, not all criteria have to be satisfied. Beams and columns, for example, are required only to demonstrate ability to carry load for the fire resistance period. Non-load bearing walls, if used as a fire separation, only have to meet the requirement of integrity and insulation (requirement that limits the temperature rise on the unexposed face). A more comprehensive discussion of the ASTM test procedure can be obtained from Lie 1992, Boring et al 1981 and Babrauskas and Williamson 1978.

2.2.2.2 Empirical Correlations

Empirical equations presenting correlation of fire resistance test results with important design parameters started to emerge when sufficient test results become available to quantify the effects of critical parameters. Some of these equations were developed on the basis of theoretical predictions, and were subsequently validated by test results.

The important points to consider in assessing the suitability of using these equations are (Lie 1992):

- The scope of the database used to validate the empirical relationships,
- The level of confidence in the calculated results, and
- The applicability of the established relationships to the specific materials and products used in actual construction.

With these words of caution, an empirical equation will be presented for illustrative purposes.

Steel Columns Protected by Gypsum Wallboard

A common protection for steel columns is to box it in using gypsum wallboard. Based on the results of accumulated fire-test data, a number of empirical equations have been developed to determine the fire resistance of columns protected by gypsum wallboard. One of them is given below: (AISI 1980, Flemington 1980)

$$R = 130\left(\frac{h\frac{W'}{D}}{2}\right)^{0.75} \tag{4.1}$$

Where:

R = fire resistance (minutes)

W' = weight of steel column and gypsum wallboard protection per foot length (lb/ft)

H = thickness of gypsum wallboard (inches)

D = developed heated perimeter (inches), which may be defined as the inside perimeter of the fire protection.

To derive the total weight W' of both the column and its gypsum wallboard protection, the following formula can be used:

$$W' = W + 50\frac{hD}{144} \tag{4.2}$$

Where W = Weight of the steel per foot (lb/ft)

More of these empirical relationships for different building materials can be found in the following references:

- Steel, concrete and timber: Lie 1992.
- Concrete and timber: NRCC 1985; Lie 1977; Lie & Allen 1972; CSA 1984
- Steel: Lie & Stanzak 1973: AISI 1980.

2.2.2.3 Theoretical Calculation

In recent years, considerable research has been undertaken with a view to developing analytical methods for calculating the fire resistance of structural elements and assemblies. Procedures are then validated against fire resistance test results. Analytical methods for determining fire resistance must consider three basic aspects of the problem:

(i) Fire characteristics
(ii) Heat transfer, and
(iii) Structural response.

The fire characteristics refer primarily to its time-temperature relationship. The time-temperature relationships for the standard fire and "real" fire are described in details in chapter 8. The other important fire characteristic is the expected fire severity, which is explained in the next section.

The heating of the structural member can be addressed using principles of convection and radiation heat transfer. Heating within the member is anlysed by conduction heat transfer. Heat transfer analyses can also be facilitated by the use of computer software. For example, FIRES-T3 and TASEF-2 are two computer programs for calculating heat transfer from fires to structures (Iding et al 1977 and Wickstrom 1979).

The structural response is determined by structural analysis calculations. The calculations are similar to those conducted for normal structural engineering purposes, except that only gravity loading (and thermal load) is considered and material properties are evaluated at elevated temperature. Computer software are also available for analysing structural responses. Examples are FIREDESIGN and FASBUS II (Anderberg 1985 and Jeanes 1985 respectively).

Typical calculations for heat transfer and structural analysis are shown in chapter 8. Some analytical methods for determining the fire resistance of building elements of various materials can be obtained from the following references:

- Steel: Milke (1995);
- Concrete: Fleischman (1995)
- Timber: White (1995)

The most important problem with regards to the use of a theoretical approach is that it requires access to a good database on material properties at elevated temperatures. Knowledge of the thermal and mechanical properties as a function of temperature is critical to the accuracy of the calculation model. Database on the effects of temperature on the thermal and mechanical properties of materials have been compiled by many researchers (e.g., Harmathy 1983) but, generally speaking, the database is incomplete. As more elevated temperature measurements are made for different construction materials, the full potential of this design approach will gain significance in practice.

2.3 FIRE SEVERITY

Fire severity is a measure of the destructive potential of a fire. For a given fire severity, a building component of relatively lower fire resistance will be "destroyed" or lose its designed function earlier than a component of relatively higher fire resistance. Fire severity is most often defined in term of a time period of exposure to the standard test fire. However, real fires have very different characteristics from the standard fire and these have resulted in several methods of determining "equivalence" to the standard fire exposure

2.3.1 Equivalent Fire Severity

The concept of "equivalent fire severity" can be used to relate the severity of an expected real fire to the standard test fire. This is important when designers estimating real fire exposure want to use published fire resistance ratings from standard tests. The various methods of comparing real fires to the standard test fire are described below.

2.3.1.1 Equal Area Concept

The initial idea on equivalent fire severity was due to Ingberg (1928) who, following a series of compartment tests with known fire loads, suggested that fire severity could be calculated by considering equivalence of areas under standard fire time temperature curve and the compartment fire curve above a base of either 150°C or 300°C as shown in Fig. 4-1

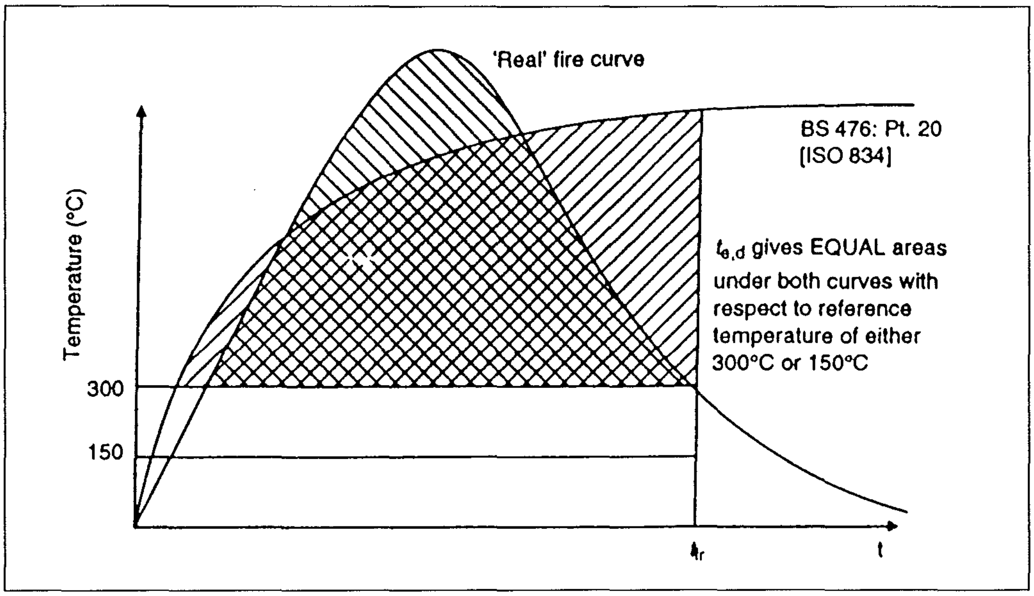


Figure 2-1 Equivalence of fire severity based on areas beneath the standard and compartment fires' time - temperature curves.

In this way, Ingberg derived a correlation between fire load measured in his tests as load per unit floor area and the standard fire resistance periods. However, this method has little theoretical justification because the unit of area is not meaningful. This method also appeared not to consider the effect of ventilation which would affect the equivalence, in that compartment temperature are affected by both the air supply and the fuel supply. This flaw led to the rejection of Ingberg's approaches and the formulation of alternative approaches

2.3.1.2 Maximum Temperature Concept

A more realistic approach, developed by Law (19710), Pettersson et al (1976) and others, is to define the equivalent fire severity as the time of exposure to the standard fire that would result in the same maximum temperature in a protected steel member as would occur in a complete burnout of the fire compartment. This concept is shown in Fig. 4.2 which compares the temperature in a protected steel beam exposed to the standard fire with those when the same beam is exposed to a particular real fire.

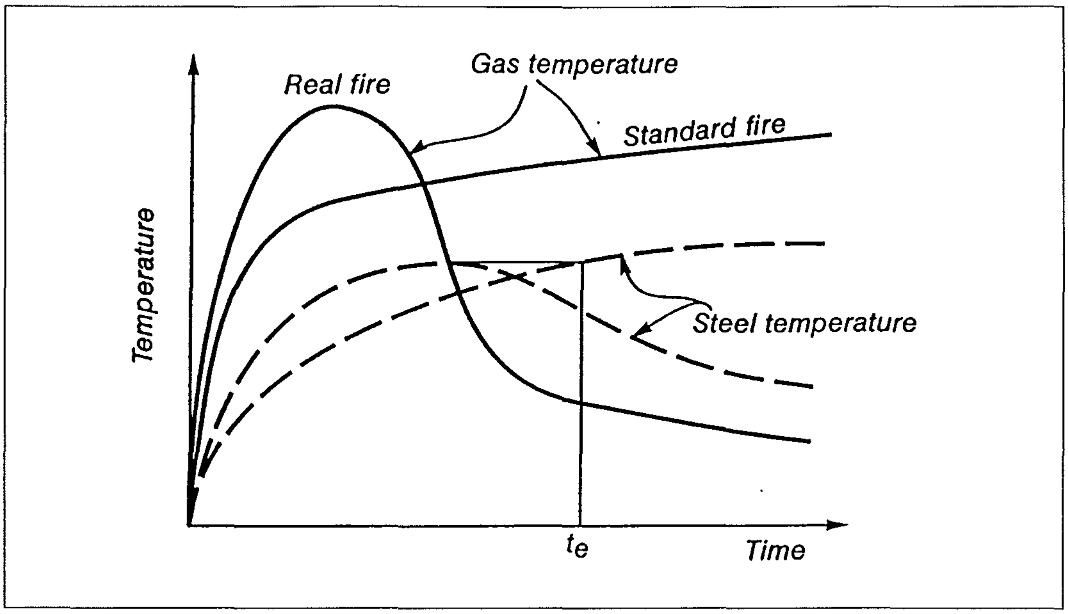


Figure 2-2 The maximum temperature concept for equivalent fire severity

In principle, this concept is applicable to insulating elements if the temperature on the unexposed face is used instead of the steel temperature, and is also applicable to materials which have a limiting temperature, such as the 300°C temperature at which charring of wood generally begins. (Buchanan 1999)

2.3.1.3 Maximum Load Capacity Concept

In a similar concept based on load capacity, the equivalent fire severity is the time of exposure to the standard fire that would result in the same load bearing capacity as the minimum which would occur in a complete burnout of the firecell. This concept is shown in figure 4.3 where the load bearing capacity of a structural member exposed to the standard fire decreases continuously, but the strength of the same member exposed to a real fire increases after the fire starts to decay and the steel temperature decreases.

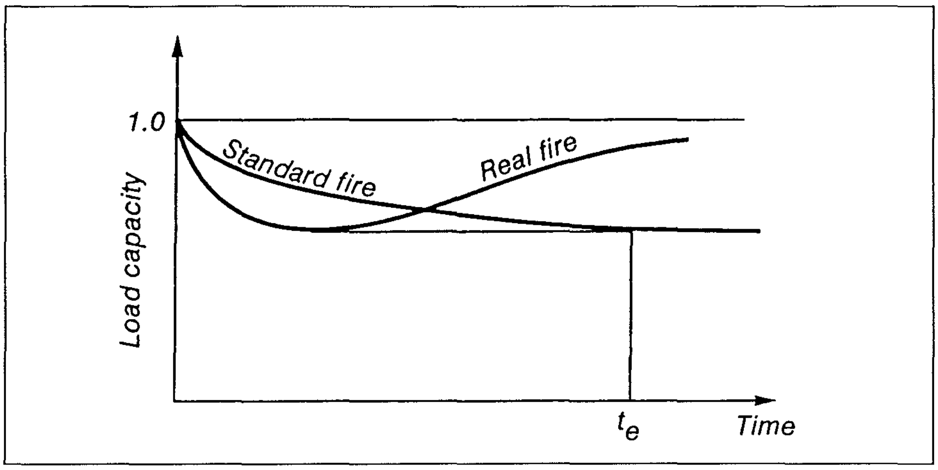


Figure 2-3 Equivalent fire severity based on load capacity concept

This concept is the most realistic for load bearing members. The minimum load concept may be difficult to implement for a material which does not have a clearly defined minimum load capacity, which may happen for wood members where charring continues after the temperature decreases (Buchanan 1999).

2.3.2 Time Equivalent Formulae

A number of time equivalent formulae have been developed by fitting empirical curves to the results of calculations of the type shown conceptually in figure 4-2 for the maximum temperature of protected steel members. Some of these formulae are described below.

2.3.2.1 CIB Formula

The most widely used time equivalent formula is that published by the CIB W 14 (CIB 1986), derived by Petterson (1973) based on the ventilation parameters of the compartment and the fuel load. The equivalent time of exposure t_e (min) to an ISO 834 test fire is given by:

$$T_e = k_e \, w \, e_f \qquad (4.3)$$

Where: e_f is the fuel load (MJ/m^2 of floor area)

k_c is the parameter to account for different compartment linings

w is the ventilation factor ($m^{-0.25}$) given by:

$$w = A_f / (A_v A_t H_v^{0.5})^{0.5} \tag{4.4}$$

Where: A_f is the floor area of the compartment (m^2)

A_v is the total window area (m^2)

A_t is the total area of the bounding surfaces of the compartment (m^2)

H_v is the height of the windows (m)

2.3.2.2 Law formula

A similar formula was developed by Margaret law on the basis of tests in small compartments (Thomas and Heselden 1972) and larger compartments (Law 1973). The formula is given by:

$$t_e = e_f A_t / [A_v(A_t - A_v)]^{0.5} \tag{4.5}$$

The CIB and the Law formulae are only valid for compartments with vertical openings in the walls. The Law formula gives similar results to the CIB formula, generally with slightly larger time equivalent values (Buchanan 1999)

2.3.2.3 Eurocode Formula

The above formulae were later modified and incorporated into the Eurocode (EC1 1994), often referred to as the "Eurocode Formula" which gives time equivalence, t_e (min), as follows:

$$t_e = k_b\, w\, e_f \tag{4.6}$$

where k_b replaces k_c and the ventilation factor w is altered to allow for horizontal roof openings. The ventilation factor is given by:

(4.7)

$$w = \left(\frac{6.0}{H_r}\right)^{0.3} \left[0.62 + \frac{90(0.4 - \alpha_v)^4}{1 + b_v \alpha_h}\right] > 0.5$$

where: H_r is the compartment height (m)

$\alpha_v = A_v/A_f \qquad 0.05 \leq \alpha_v \leq 0.25$

$\alpha_h = A_h/A_f \qquad \alpha_h \leq 0.20$

$b_v = 12.5(1+10\alpha_v - \alpha_c^2)$

A_f is the floor area of the compartment (m^2)

A_v is the area of vertical openings (m^2)

A_h is the area of horizontal openings (m^2)

An important difference from the CIB formula is that the Eurocode equivalent time is independent of opening height, but depends on the ceiling height of the compartment, so the two formulae can give different results for the same room geometry. The results are similar for small compartments with tall windows, but the Eurocode formula gives much lower severities for large compartments with tall ceilings and low window heights (Buchanan 1999).

Values of the terms k_c and k_b are given in Table 4.1, where they are shown to depend on the compartment materials (roughly inversely proportional to the thermal inertia).

Table 2-1 Values of k_c and k_b used in time equivalence formulae

			$\sqrt{(k\ \rho\ c_p)}$			
			High	Medium	Low	
Formula	Term	Units	> 2500	720 - 2500	< 720	General
CIB W14	k_c	Min $m^{2.25}$/MJ	.05	.07	.09	.10
Eurocode	k_b	Min m^2/MJ	.04	.055	.07	.07
k=thermal conductivity (W/mK) ρ = density (kg/m^3) c_p= specific heat (J/kgK)						

The "general" case is that recommended for compartments with unknown materials. It should be noted that k_c and k_b have slightly different numerical values and dimension, because of the different ventilation factors in the respective formulae.

Using typical thermal properties of the materials from Table 4-1, a building constructed from steel is in the "high" category, normal and lightweight concrete are in the "medium" category, gypsum plaster and any materials with better insulating properties are in the "low" category.

2.3.3 Choice of Formula in This Project

The Eurocode formula is used in the Approved Documents to the New Zealand Building Code published by the Building Industry Authority (BIA 1992). It will thus be the choice of formula for assessment in this project.

3 STRUCTURAL FIRE DESIGN – AN OVERVIEW

3.1 DEFINITION

Structural fire design is concerned mainly with the prevention of fire spread through separating vertical and horizontal partitions (compartmentation) and the avoidance or limitation of structural failure or damage - referring to fires which fail to be controlled at an early stage. The basic unit for structural fire design is the fire compartment or fire zone (CIB 1986).

3.2 LEVEL OF STRUCTURAL FIRE SAFETY

The level of structural fire safety to be provided by design should be governed by:

(a) the risk to life and property in the case of a severe fire considered as an accidental situation;

(b) the risk-reducing effect of structural measures, depending on the following:

   (i) Type (height) of building and its use,

   (ii) Occupancy of the fire compartment,

   (iii)Function of the various structural components.

(c) the risk-reducing effect of non-structural measures for fire risk control should be considered, in particular in terms of reduced frequency of severe fires that may result.

3.3 STRUCTURAL FIRE REQUIREMENT-WHOLE STRUCTURE

Building structures should be designed, constructed and maintained so that they display an acceptable performance and fulfil specified functions in the event of fire (CIB 1986). These functions are explained below.

3.3.1 Load Bearing Capacity

The load bearing structure should adequately withstand all actions - including temperature, loads and imposed deformations - during fire exposure. This means that the individual members have sufficient resistance capacity and that local failure will not necessarily entail collapse or instability of the whole structure or any major subassembly. At the same time, the total structure should have sufficient stability and ductility.

3.3.2 Separating Function

A fire compartment should adequately confine a fire to a limited area. Thus all vertical and horizontal partition surrounding a fire compartment should fulfil a specified separating function in terms of providing for sufficient thermal insulation and displaying sufficient integrity during fire exposure.

3.3.3 Repairability

Whilst designing for sufficient load bearing capacity generally ensures that the structure is not likely to collapse in the event of fire, it may be damaged to an extent requiring some degree of demolition and reconstruction. Hence repairability of the structure at reasonable cost may be required.

3.3.4 Reserviceability

In certain cases reserviceability of the structure after fire may be required. Reserviceability implies a limitation of damage to an extent necessitating only a short (or no) interruption of use of the building for repair.

3.4 STRUCTURAL FIRE REQUIREMENT-INDIVIDUAL MEMBER

The main requirement for individual structural element is the fire resistance. The meaning of fire resistance, fire resistance rating and method of assessment have been covered in section 2.2.

3.5 PRINCIPLES OF STRUCTURAL FIRE DESIGN

Structural fire design generally consists of the following:-

(a) Assessment of the heat exposure and structural response, according to methods which are established for the type of material or construction;

(b) Structural detailing which involves an adequate choice of the structural system, the geometry of the structure and its various components including supports, joints etc according to rules given in the relevant codes, standards and specifications;

(c) Material detailing, i.e., an adequate choice of materials with specified thermal and mechanical properties, according to rules given in the relevant material related documents.

Fire design should be in accordance with the state of engineering knowledge in this field. Design should be based on fire exposure to be expected under the conditions, either

- representative for the certain types of buildings and occupancy, or
- relevant for the particular application or use.

Design verification should consider the frequency of fires, their expected severity, the nature of the thermal and structural response and the actions relevant in fire exposure as well as any model uncertainties.

3.6 STRUCTURAL ANALYSIS FOR FIRE CONDITION

Structural analysis for fire is conceptually similar to structural design for normal temperature conditions. The design can be carried out in either the serviceability limit sate or ultimate limit state format. The main differences of fire design compared with normal temperature design are that at the time of fire (Buchanan 1999):

- the applied loads are less
- strength of materials may be reduced by elevated temperatures

- cross section areas may be reduced by charring or spalling
- smaller safety factors may be used, because of the low likelihood of the event
- deflections are not important (unless they affect strength)
- different failure mechanisms need to be considered

4 STRUCTURAL STABILITY DURING FIRE

4.1 INTRODUCTION

As the title "Structural Stability during Fire" implies, this section addresses the design of the structural system of the a building in such a way that its primary load bearing members will not collapse prematurely in the event of a fire. Important aspects of this section include the regulatory requirements, specific fire safety objectives that can be pursued by providing structural stability, a discussion on how much fire resistance is required to be designed for to fulfil this objective, and what means are available to provide the necessary level of fire resistance.

4.2 REGULATORY FRAMEWORK

In New Zealand, the First Schedule to the Building Regulations 1992 is based on the Building Act 1991 and contains 35 working clauses. The clauses dealing with fire are:

C1: Outbreak of fire
C2: Means of escape
C3: Spread of fire
C4: Structural stability during fire

Each of these clauses is presented in the BIA handbook to the New Zealand Building Code complete with its relevant legal provisions, namely its objective, functional requirement and performance statements (BIA 1992). The clause of interest here, clause C4: Structural stability during fire is appended below:

Clause C4 - **STRUCTURAL STABILITY DURING FIRE**

Objective

C4.1 The objective of this provision is to:

(a) Safeguard people from injury due to loss of structural stability during *fire*, and

(b) Protect *household units* and *other property* from damage due to structural instability caused by *fire*

Functional Requirement

C4.2 *Buildings* shall be constructed to maintain structural stability during *fire* to:

(a) allow people *adequate* time to evacuate safely,

(b) allow fire service personnel *adequate* time to undertake rescue and fire fighting operations, and

(c) avoid collapse and consequential damage to adjacent *house units* or *other property*.

Performance

C4.3.1 Structural elements of *buildings* shall have *fire* resistance appropriate to the function of the element, the *fire load*, the *fire intensity*, the *fire hazard*, and the height of the *buildings* and the fire control facilities external to and within them.

C4.3.2 Structural elements shall have a *fire* resistance of no less than that of any element to which they provide support within the same *firecell.*

C4.3.3 Collapse of elements having lesser *fire* resistance shall not cause the consequential collapse of elements required to have a higher *fire* resistance

The Building Industry Authority has dealt with each clause, presenting it as an Approved Documents providing non-mandatory guidance to a single method of compliance with the law (BIA 1995). Each Approved Document includes either an analytical Verification Method or a prescriptive Acceptable Solution, or both. A Verification Method is offered in the case of clause C1 but not for C2, C3 or C4. For each of the latter an Acceptable Solution is given. The Acceptable Solution for clause C4 is summarised below:

Primary element loadings

- To provide for structural stability of primary elements, they should be fire rated to avoid premature failure

- During a fire primary elements shall resist collapse under:
  a) The design dead and live loads required by NZS 4203 (SNZ 1992)
  b) Any additional load caused by the fire

Non-evacuation

- Special attention shall be given to situations where evacuation is either not possible or not desirable such as in buildings which:
  a) contain purpose groups SC (sleeping when in care of others) and SD (sleeping when in legal detention) or
  b) Are required to function continuously during an emergency, e.g., operations rooms in a civil defence headquarters or police station.
- In such situations the accommodation concerned, the services to it, and the means of escape, shall remain safe for the duration of the a fully developed fire in an adjacent firecell.

General principles

- Factors influencing the necessary level of fire resistance include:
  a) Fire severity
  b) Building height
  c) Total fire load
  d) Purpose group
  e) Occupant load
  f) Capability of Local Fire service
  g) Availability of a water supply
  h) Level of fire safety precautions installed in the building
- Primary elements are always rated for stability, but in some cases will also need to be rated for integrity and insulation.

In the performance requirement C4.3.1 the word "appropriate to the function of the element" means that the required level of performance (degree of fire resistance) will vary according to design circumstances. With current technology, the collapse of a member subjected to the effect of a fully developed fire may be delayed a few minutes, a few hours or indefinitely. Delaying collapse indefinitely is not always necessary. For example, small buildings from which occupants can escape rapidly in the event of a fire may not require any special protection against collapse and their design will satisfy the objective of life safety. Whether structural fire protection should be provided for these buildings, and to what extent it should be provided, becomes a decision that is based strictly on the economics of the situation.

In a larger building, or one where occupants may have difficulty evacuating, the time required for a complete evacuation will be longer. Also, fire fighters will be expected to enter the building, assist in the evacuation and attempt to extinguish the fire. Collapse may be delayed by providing a higher level of structural fire protection in order to ensure the life safety of occupants and fire fighters. Again the level of protection above the minimum needed to satisfy the life safety objective will depend on economics.

When the height or occupancy of the building is such that total evacuation is not possible, collapse of the primary structural members must be prevented for the period of emergency, and beyond.

4.3 METHODS OF INCREASING FIRE RESISTANCE

Structural system can be made more fire resistant by increasing the member sizes (structural over-design), by encasing the structural elements in an insulating material of low thermal inertia, or by protecting the entire assembly or system with an insulating membrane. The type of protection best suited for a particular system depends primarily on the type of material used in its construction, as each material behaves differently under elevated temperatures. A brief description of the important design parameters affecting fire resistance, as well as the most common types of protection for concrete, steel and timber structural system follows.

4.3.1 Concrete Construction

Reinforced and prestressed concrete systems are rarely protected externally, since concrete is normally made of inorganic materials having low thermal conductivity and heat capacity. However, concrete gradually loses its compressive strength under increasing temperatures so that it is necessary to ensure that the members have been designed with sufficient reserve strength to support the applied loads for the projected duration of the fire exposure.

Another important design consideration consists of making sure the steel reinforcement is sufficiently insulated, since steel loses considerable tensile strength at elevated temperatures. The critical temperature of steel is defined as the temperature at which only 60% of the original strength remains, at which point failure is imminent under full design loads. For regular reinforcing steel, the critical temperature is 538°C, whereas for prestressing steel bars, which are made of high carbon, cold drawn steel instead of low carbon, hot-rolled steel, the critical temperature is significantly lower at 427°C (Fitzgerald 1997). The time it will take for these temperature to be reached in concrete members (slab, beam or column) depends on the thickness of the concrete cover protecting the steel.

The degree of restraint against thermal expansion, which every concrete member will undergo as its temperature increases, and the degree of continuity provided by the structural system at the supports, will also affect the fire resistance. Both are generally regarded as being beneficial insofar as concrete members are concerned. Restraint against expansion sets up additional compressive stresses which, when accounted for in the design, reduce the tensile forces that are initially resisted by the reinforcing steel in the bottom half of the member. Continuity enables a certain amount of stress redistribution to take place before excessive rotation develops at the supports and mid-span, causing the collapse of the assembly.

4.3.2 Steel Construction

Steel, like concrete, has the advantage of being noncombustible, but this characteristic alone means little in trying to resist collapse. Its high thermal conductivity makes steel absorbs heat much more quickly than other materials; thus if the structural member has a relatively small mass, its temperature will increases very rapidly. Both the yield stress and the modulus of elasticity, the two material properties most important in determining load-carrying capacity, decrease considerably with increasing temperature (DeFalco 1974)). At a temperature of 593°C, these values will have fallen by at least 40% compared to ambient room temperature levels. This means that the strength of the steel member will be barely sufficient to resist applied loads (assuming normal safety factors).

The mass-to -heated perimeter ratio for a steel structural member is a good indicator of its inherent fire resistance. A heavy steel column can absorb considerable heat and not reach its critical temperature before 30 to 40 minutes of exposure to a fully developed fire. On the other hand, open web steel joints and other light weight types of steel construction may fail within 5 to 10 minutes of exposure to the same fire.

In order to achieve fire resistance ratings of one hour or more, a steel member must be protected by an insulating skin that will keep its temperature below the critical temperature. Encasement in concrete, brick, clay tiles, lath and plaster, and similar material used to be the common methods. Now, less expensive forms of protection have been developed, such as cementitious coatings and sprayed on mineral fibres, which can be applied directly on the steel members. Some intumescent paints and epoxy coatings have also been used to improve the fire resistance of steel members. These coating swells and form an insulating layer around the members when subjected to heat.

As an alternative to encasement and surface treatments, the use of a suspended ceiling or protective membrane has been a common means of protecting steel floor and roof assemblies (Read & Adams 1979). The membrane usually contributes between 85 to 90% of the fire resistance of such assemblies; thus the type, thickness and fastening of the membrane are the most important design parameters. Lath and plaster, gypsum wallboard panels and inorganic acoustical tiles supported on a metallic-grid system are the most popular membrane ceilings. They can be attached directly to the underside of steel framing members or hung with wire hangers from either the floor deck or the framing members. This allows flexibility in the depth of the concealed space, which is often used for electrical, plumbing and mechanical services. For these protective ceilings to be effective, however, it is critical that all services penetrations be adequately protected or fire stopped and that the integrity of the membrane not be destroyed during maintenance work in the concealed space. Also, when acoustical tiles are used on a metal grid system, they must be tied down with special clips so that they will not be uplifted by the positive pressure which may be created by a fire.

A final aspect to consider when using steel in construction is its significant coefficient of linear expansion under elevated temperature. If the structural member is axially restrained against displacement (as column is), the expansion due to heat will be translated into thermal stress that will increase the overall stress level in the member and cause an earlier collapse. Without axial restraint, a steel member will expand and could set up eccentric loading of adjacent structural member by displacing one of their ends (for example, a beam displacing the top of a column or of a load bearing masonry wall). Good fire protection engineering dictates that either thermal expansion be prevented by limiting steel temperature, or its effect on the structure be accommodated in the design.

4.3.3 Timber Construction

Wood has the major disadvantage of being combustible. However, this does not mean that wood construction is less safe than steel or even concrete construction. The burning of wood produces a charred layer on the surface of a member, which acts to insulate the unburned wood from the heat being radiated by the flame. This slows down considerably the rate of charring, which will be relatively constant throughout the fire. The rate of charring varies according to the wood species and moisture content but is roughly 0.65mm/min for thick members (SNZ 1993b).

Wood also has a very low thermal conductivity, which means that the inside of a wood member is little affected while the outside surfaces burn. In fact a correlation exists between the strength of a burning member and its reduced cross-sectional area. The charred layer is presumed to have no strength, while the uncharred wood suffers a strength loss of only 10-15% (Lie, 1972). The term "heavy timber construction" refers to a combustible type of construction that utilises the property that large members which can burn for a significant period of time before their cross-sectional areas are reduced to the point of collapse. By carefully specifying minimum sizes for each critical component, building codes often consider heavy timber construction as equivalent to three quarter-hour rated combustible construction (Purkiss 1996).

As with steel construction, wood floors and roof systems can acquire considerable fire resistance if the wood framing members are protected with an insulating membrane of lath and plaster or gypsum wallboard. The membrane can be fastened directly into the wood frame, provided the fasteners are sufficiently long to dissipate heat into the wood and prevent local charring that might cause them to pull out. Wood columns can be protected by intumescent coatings, gypsum board, or other mineral fibreboard developed specifically for the purpose. Although fire retardant treatments (surface applied or pressure impregnated) may delay ignition and therefore slow down the spread of flame along the treated wood members, they generally do not affect the rate of charring under fully-developed fire conditions and, hence, they do not increase the fire resistance of wood members.

4.4 APPROACHES FOR ASSESSING STRUCTURAL ADEQUACY UNDER FIRE

Any approach for assessing the structural adequacy of a member or structure under fire conditions must address three matters (I E A 1989):

(a) the temperature history of structural members resulting from the relevant fire situations,

(b) the influence of this temperature history on the structural behaviour of a member or structure, and

(c) the loads on the structure or member.

Temperature history of a member can be determined by calculation or test, and various combinations of test data and calculations are evident in the numerous methods that have been proposed for obtaining the time-temperature curves of members exposed to various fires. (Buchanan 1999)

Given the temperature history of the structural member, the relevant mechanical properties can be modified for the effects of temperature. Anderberg (1976) has conducted an extensive research program on the behaviour of concrete under elevated temperature conditions and much is known about the influence of temperature on the mechanical properties of steel (Bennetts et al 1981). In the case of timber, it is common to ignore the material affected by the fire and base structural assessment on the remaining timber section with normal temperature properties (Lie, 1972)

The combination of fire and other extreme loads is seen as an unlikely event. The relevant live load in fire is therefore taken as the arbitrary point-in -time live load and not the extreme or peak value. Wind, snow and seismic loads are not included in the analysis.

The various approaches for assessing structural adequacy under fire condition are briefly outlined below (I E Aust 1989):

Approach 1 - Deemed-to-Comply (Member Performance)

This consists of specifying standard dimensions to achieve a required fire-resistance performance as measured by standard fire-test conditions. Examples of this approach are shown in Table 2.1, where the term FRL corresponds to fire resistance level. For the purpose of these examples FRL refers to the minutes with respect to structural adequacy deemed to be achieved for the constructions in a standard fire test.

Table 4-1 Examples of 'Deemed-to-comply' Approach for Structural Adequacy

From I. E .A. 1989

Structural Member	Construction of Members	Thickness of Principal Material (mm)		
		FRL 60 min	FRL 90 min	FRL 120 min
(i) Non load bearing wall	Unplastered clay brick	100		
(ii) Steel column	Gypsum perlite plaster	22	25	35

Deemed-to-comply methods have been employed in the various building codes for many years and have the distinct advantage of simplicity. The deemed-to-comply geometries are based on available fire test data and, in many cases, guessworks. The disadvantage with this approach is that it is not well-based and sometimes can be unconservative.

Approach 2 - Standard Fire- Test and Variation (Member Performance)

Another approach is to conduct a standard fire test on a member identical to the prototype in every way. The tested member must be supported and loaded in a conservative manner compared with the prototype. Although standard fire-tests must be conducted, this approach is very restrictive, as it is unlikely that a test conducted on a given member will be appropriate to other structural members.

Approach 3 - Fire Test Series and Structural Engineering Calculations (Member Performance)

In this method, a series of fire tests are conducted and the test results are interpreted using an engineering analysis with regard to both the thermal and structural response. The advantage of the method is that it allows economic use of fire test data in that once a minimum number of tests have been conducted, other cases not covered by the tests can be handled by means of interpolation.

Approach 4 - Standard Fire-Test Simulation: Calculation Based on Assumed Properties (Member Performance)

For structural members constructed of materials such as masonry, steel, and concrete, it is possible to calculate theoretically the performance of a member in a standard fire-test. This approach has been used to provide the basis of design rules for reinforced concrete and composite members. To be effective, it is important that conservative, but representative, thermal and mechanical properties have been established for elevated temperature conditions. This has certainly being done for concrete, steel and masonry. Such material are inherently more predictable, from a thermal performance point of view, than some forms of lightweight fire protection in which complex internal reactions take place during heating. Once the thermal behaviour of the structural member under simulated standard fire-test conditions has been established, the structural behaviour can be computed using the appropriate methods.

Approach 5 - Standard Fire Test Simulation - Calculation Based on Assumed Properties (Structure Performance)

This approach is identical to Approach 4 but relates to the behaviour of a building structure in that it would enable evaluation of the structural adequacy of an entire building with fire in one compartment. The American Iron and Steel Institute (AISI) has used this approach for several large building project in the United States by using the program FASBUS.

It needs to be emphasised that such an approach goes well beyond the current regulatory approval system, which looks only at isolated members and not at their interaction. The cost and time involved in undertaking such analysis will considerably be greater (and more complex) than the normal analysis of the structure. Although useful for certain isolated cases and as a research tool, this approach should not be considered for most situations.

Approach 6 - Standard Fire-Test - Equivalent Time Method (Member Performance)

All the above methods have been related to the standard fire-test or the heating environment associated with it. The equivalent time have been proposed by Odeen (1970), CIB (1986) and more recently by Eurocode (EC1 1994) as a means of relating a "real" fire environment back to an equivalent exposure under standard fire-test conditions. Equivalent time is explained in section 3.3 of this report.

Once the equivalent fire-exposure has been determined, the member can be assessed using any appropriate structural methods.

Approach 7 - "Real" Fire Analysis (Structure Performance)

As with approach 6, this method can be applied to fire compartments in buildings with specified occupancies. In particular, it is typified by that given by Pettersson et al (1976) where "real" fire temperature-time curves have been prepared for compartments with various degrees of ventilation, fire load, and broad categories of lining materials. A more recent representation of the "real" fire is the "parametric" fire formula proposed by Eurocode (EC1 1994). Once the fire temperature-time response has been established, the structural behaviour can be determined using the appropriate structural methods.

5 UNCERTAINTIES IN STRUCTURAL FIRE DESIGN

5.1 INTRODUCTION

The main aim of structural fire engineering design is the assurance of structural performance under fire conditions. However, most planning and design of buildings to be structurally safe under fire condition must be accomplished without the benefit of complete information; consequently the assurance of performance cannot be fully guaranteed. Many decisions that are made during the process of planning and design are invariably made under conditions of uncertainty. All quantities (except physical and mathematical constants) that currently enter into engineering calculations are in reality associated with some uncertainty. This fact has been implicitly recognised in current and previous codes. If this were not the case, a safety factor of only slightly in excess of unity would suffice in all circumstances. Therefore, under fire situation, there is invariably some probability of non-performance or failure of the building structure together with its associated consequences. The determination of appropriate standards of safety requires the quantification of these uncertainties by some appropriate means and a study of their interaction for the structure under consideration. It is also now widely recognised that some risk of structural failure must be tolerated (Hart, 1982). The main object of structural fire design is therefore to ensure, at an acceptable level of probability, that the structure will at least remain stable under a fully developed fire for sufficient time to allow the occupants time to escape and fire service personnel to undertake rescue or firefighting activities.

5.2 UNCERTAINTY IN ENGINEERING DESIGN

In general, structural reliability analysis is concerned with the rational treatment of uncertainties in structural engineering design and the associated problem of rational decision making. Uncertainties need to be identified and classified so that their relevance to the problem at hand can be ascertained. Before doing that it is helpful to introduce the concept of basic variables.

5.2.1 Basic Variables

For the purpose of quantifying uncertainties in the field of structural fire engineering and for subsequent reliability analysis it is necessary to define a set of basic variables. These are defined as the set of basic quantities governing the static or dynamic response of the structure. Basic variables are quantities such as mechanical properties of materials (e.g. yield strength), geometrical quantities (e.g. section modulus), unit weights, external loads (e.g. dead load), etc. They are basic in the sense that they are the most fundamental quantities normally recognised and used by designers in structural calculations. Thus, yield stress of steel can be considered as a basic variable, although this property is itself dependent on chemical composition and various micro-structural parameters. Ideally, basic variable should be chosen so that they are statistically independent quantities. However, this may not be possible if the strength of the structure is known to be dependent on, for example, any two mechanical properties that are known to be correlated, e.g. the tensile strength and compressive strength of a batch of concrete.

5.2.2 Types of Uncertainty

In general, a wide range of uncertainties needs to be considered in structural fire engineering. These might include various environmental conditions, workmanship and human error, and prediction of future events. Various quantitative techniques are available for systematic identification of uncertainties. These include "event-tree analysis" (Henley and Kumamoto 1981) and "hazard scenario analysis" (Schneider,1981). More generally, techniques such as "brainstorming" (Osborn,1957) may be used. All techniques amount essentially to a critical analysis of the problem to be analysed, consideration of all imaginable consequences and all imaginable possibilities and retaining only those with some finite probability of occurrence. Further, all techniques rely on having available expert opinion for the various assessments to be made and up-to-date information on which to base these assessments.

To illustrate where and how different types of uncertainty can arise in man's attempt to understand and make use of a natural phenomenon (e.g., fire) to serve his goal(s) the schematic diagram shown in Fig. 4.1 (Melchers 1987) will be referred to.

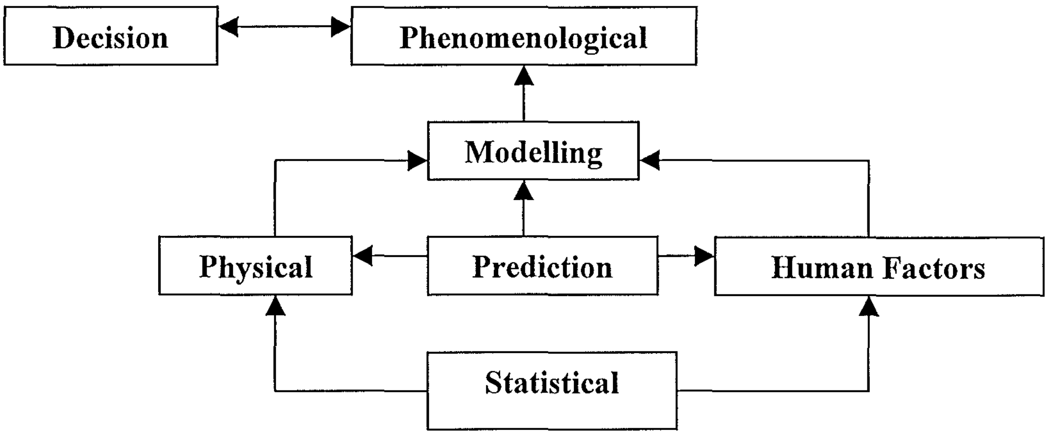


Figure 5-1 Interrelationship of Uncertainties in Reliability Assessment

Source: Melchers 1987

Figure 4-1 shows a situation where a fire engineer, say, is required to undertake a unique fire engineering design. To do this he or she has to try to know and understand as much as possible about the fire phenomenon. In a scientific approach to his problem he would set up a model that could approximate the effects of the phenomenon. For the model to be accurate he has to rely on his own observation of the physical aspects of the task at hand and to make his own prediction on any aspect that is not apparent from observation. The accuracy of his modelling would be enhanced if he can make use of similar observations carried out by fellow human beings - that is, the available statistical data. The accuracy or reliability of such data depends, of course, on the human factors. The various aspects are dynamically interrelated and uncertainties definitely arise in each area. These uncertainties are further explained below for the case of structural fire design.

5.2.2.1 Phenomenological Uncertainty

Fire is certainly a phenomenon that has greatly impacted on man since his earliest existence on earth. Man continues to live in awe of fire not so much because of the various ways in which energy from fire can be harnessed to serve him but more because of the destruction fire has wrought upon him. The development of fire science has accelerated over the last 150 years. It is a complex area involving many disciplines, but it is relatively primitive compared to other technological fields (Quintiere 1998). An example of this uncertainty is in the field of fire engineering, and specifically in structural fire design, where the effect of fire on the behaviour of the structural element is not yet fully understood or quantifiable. The effect on the whole structure is even less understood. The bulk of the knowledge on structural response due to fire is largely empirical and is based on many assumptions and hypotheses.

5.2.2.2 Physical Uncertainty

Structural responses of a whole structure or components under fire condition depend in part on the material properties at elevated temperature. These properties are not known exactly and this gives rise to physical uncertainty. Physical uncertainty is that identified with the inherent random nature of a basic variable. Specific examples include;

- The physical dimension of a structural member at elevated temperature
- The fire temperature
- Variation of the yield strength with temperature
- Variability in the actual gravity loading during fire situation

Physical uncertainty can be reduced but not eliminated with greater availability of data, or greater effort in quality control. It is a "fundamental" property of the basic variable. The physical uncertainty for any basic variable is generally not known *a piori* and must be estimated from observations of the variable or be subjectively assessed.

5.2.2.3 Statistical Uncertainty

In most cases of engineering design, values of material properties used in calculation are inferred from statistical analyses of sample observations. Data may be collected for the purpose of building a probabilistic model of the physical variability of a property. This will involve, firstly the selection of an appropriate probability distribution type, and then the determination of numerical values for the parameters. Common probability distributions have between one and four parameters which immediately places a lower bound on the sample size required. But in practice, very large sample sizes are required to establish reliable estimates of the numerical values of the parameters. Therefore, for a given set of data the distribution parameters may themselves be considered as random variables, the uncertainty of which is dependent on the amount of sample data or any prior knowledge. This uncertainty is termed statistical uncertainty and, unlike physical uncertainty, arises solely as a result of lack of information. An example of such uncertainty in building construction for fire safety is the fire resistance rating of building components like pre-cast concrete beams and columns.

5.2.2.4 Modelling Uncertainty

Structural fire design and analysis sometime make use of mathematical models relating desired output quantities (e.g, the temperature of steel member) to the values of a set of input quantities or basic variables (e.g fuel load density, ventilation factors). These models are generally deterministic in form. Furthermore, they may be based on an intimate understanding of the mechanics of the problem (e.g. modes of heat transfer) or they may be highly empirical (e.g. parametric time-temperature relationship). However, with very few exceptions, it is rarely possible to make highly accurate predictions about the structural response of both the components and the whole structure under fire condition. In other words, the response of structural elements to fire and loading under fire condition contains a component of uncertainty in addition to those components arising from uncertainties in the values of the basic loading and strength variables. This additional source of uncertainty is termed modelling uncertainty and occurs as a result of simplifying assumptions, unknown boundary conditions and also as a result of the unknown effects of other variables and their interaction which are not included in the model. In many components and structures, model uncertainties have a large effect on structural reliability and should not be neglected (Thoft-Christensen & Baker 1982).

5.2.2.5 Prediction Uncertainty

Structural fire design involves the prediction of a future state of affair; for example, the prediction of the probability of fire occurring and the resulting structural response. The soundness of a prediction depends on the state of the available knowledge. As new knowledge related to the structural response under fire condition becomes available, the prediction and hence the design will become more refined, with, usually but not necessarily, a concomitant reduction in uncertainty. In other words, the accuracy of any prediction made is dependent not only on the properties of the structure, but also on the designer's knowledge of the structure and the forces and influences likely to act on it under fire condition. Similarly, if the structure is designed for a specific lifetime, the designer's uncertainty in the prediction of the structure's lifetime and the peak loading during this lifetime also add additional uncertainty to the overall process.

5.2.2.6 Decision Uncertainty

In structural fire design, or for that matter any design or undertaking, a series of decisions have to be made. Sound or correct decisions depend primarily on the elimination or reduction of all or most of the uncertainties described earlier. However, uncertainty will still arise in the decision making. An example of decision uncertainty is in connection with the decision whether, for example, a limit state has been violated. In the aftermath of a fire the engineer has to decide, based on certainty of his own engineering judgement or experience, whether the fire damaged structure is repairable, still serviceable or has violated the ultimate limit state. Another example is the choice of failure criterion for structural member exposed to fire - whether failure analysis should be done in the temperature, load bearing or time domains.

5.2.2.7 Human Factors

Arguably, it can be said that the greatest source of uncertainty in the design, construction, operation or usage, and maintenance of any engineering system comes from the "human factor". The uncertainty resulting from human involvement in engineering system normally manifest itself when the system fails and human error is determined to be the main cause. For example, human error causes 20-90% of all major system failures or accidents as data in Table 4-1 will illustrate (Stewart & Melchers 1997).

Table 5-1 Proportion of System Failure due to Human Error

System	Percentage of failures/accident	Source
Aircraft	60-70%	Christensen and Howard (1981)
Air Traffic Control	90%	Kinney et al. (1977)
Building and bridges	75%	Matousek and Schneider (1977)
Dams	75%	Loss and Kennett (1987)
Missiles	20-53%	Christensen And Howard (1981)
Off-shore platforms	80%	Bea (1989)
Power plants:		
Fossil-fuel	20%	Finnegan et al (1980)
Nuclear	46%	Scott and Gallaher (1979)
Shipping	80%	Gardenier (1981)

(From Stewart & Melchers 1997)

Human error can be roughly classified into (a) gross human error and (b) error due to human variability (Melchers 1987).

Gross human errors are a direct result of ignorance or oversight of fundamental structural or service requirement and is related more to the individual. The various factors associated with this category is shown in Table 4.2

Table 5-2 Error Factors in Observed Failure Cases (Melchers 1987)

Factor	%
Ignorance, carelessness, negligence	35
Forgetfulness, errors, mistakes	9
Reliance on others without sufficient control	6
Underestimation of influences	13
Insufficient knowledge	25
Objectively unknown situations (unimaginable ?)	4
Remaining	8

Adapted from Matousek and Schneider (1977)

Human error due to human variability is due to natural variation among the individuals with respect to ability, task performance and so on. The prime "causes" are shown in Table 4-3.

Table 5-3 Prime "Causes" of Failure (Melchers 1987)

Cause	%
Inadequate appreciation of loading conditions or structural behavior	43
Mistakes in drawings or calculation	7
Inadequate information in contract documents or instructions	4
Contravention of requirement in contract documents or instructions	9
Inadequate execution of erection procedure	13
Unforeseeable misuse, abuse and/or sabotage, catastrophe, deterioration (partly "unimaginable" ?)	7
Random variations in loading, structure, materials, workmanship, etc	10
Others	7

Adapted from Walker (1981)

The overriding from these surveys is that human error is involved in the majority of cases of recorded failure. Human error must, it seems, be considered if a reliability assessment is to relate to reality. But, because of its complexity, human behaviour cannot yet be related to all the various factors that influence it.

5.3 OVERVIEW OF UNCERTAINTY ANALYSIS

The existence of various uncertainties, particularly in the inherent randomness of fire behaviour, is the main contributor to potential failure of structural fire design. The primary objective of uncertainty analysis is to assess the statistical properties, such as the probability density function (PDF) and the statistical moments of a quantity subject to uncertainty. The knowledge of such statistical information is essential in reliability analysis. In other words, uncertainty analysis is a prerequisite for reliability analysis.

5.3.1 Different Measures of Uncertainties

Several expressions have been proposed to describe the degree of uncertainty of a parameter, a function, a model, or a system. The latter three usually depend on a number of parameters. Therefore their uncertainty is a weighted combination of the uncertainties of the contributing parameters.

The most complete and ideal description of uncertainty is the probability density function (PDF) of the quantity subject to uncertainty. However, in most practical problems such a probability function cannot be derived or found precisely (Ang & Tang 1975).

Another method of expressing the uncertainty of a quantity is to describe it in terms of a reliability domain such as the confidence interval. The methods of evaluating the confidence interval of a parameter on the basis of data samples are well known and can be found in standard statistics and probability reference books (e.g. Benjamin & Cornell 1970). Nevertheless, this method of confidence interval has a few drawbacks, including (a) the parameter population may not be normally distributed as assumed in the conventional procedures to determine the confidence interval, and this problem is important when the sample size is small; (b) there exists no means to combine directly the confidence intervals of individual contributing random components to give the confidence interval of the system as a whole (Ang & Tang 1984).

A useful alternative is to quantify the level of uncertainty is to use the statistical moments of the random variable. In particular, the second moment is a measure of the dispersion of a random variable, and either the variance or standard deviation can be used. The coefficient of variation, which is the ratio of the standard deviation to the mean offers a normalised measure useful and convenient for comparison and for combining uncertainties of different variables.

5.3.2 Methods of Uncertainty Analysis

Methods for performing uncertainty analysis vary in different level of sophistication. They are also dictated by the information available regarding the stochastic input parameters. In principle, it would be most ideal to derive the exact probability distribution of the model output as function of those of the stochastic input parameters. However, most of the models or design procedures used in fire engineering are nonlinear and highly complex. This prohibits any attempt to derive the probability distribution of model output analytically. As a practical alternative, engineers frequently resort to methods that yield approximations to the statistical properties of uncertain model outputs. Some of the methods that can and have been used in uncertainty analysis are very briefly mentioned below.

5.3.2.1 First-Order Variance Estimation Method

This method estimates uncertainty in terms of the variance of system output using the variances of the individual contributing factors. This method needs only the mean and standard deviation of the stochastic parameters involved in engineering models. Complete description of this method can be found in standard texts (e.g., Ang & Tang 1975, Benjamin &Cornell 1970).

5.3.2.2 Rosenblueth's Method

Rosenblueth's (1975) method is based on a Taylor series expansion about the means of input variables. It can be used to estimate statistical moments of any order of a model output involving several stochastic input variables which are either correlated or uncorrelated.

5.3.2.3 Integral Transformation Techniques

The well known integral transforms are Fourier transform, Laplace transform and exponential transform. Use of the integral transform methods for uncertainty analysis requires knowledge about the PDF's of stochastic input parameters in the model. Tables of various integral transforms of functions can be found in many mathematical handbooks (e.g, Abramowitz and Stegun, 1972).

5.3.2.4 Monte Carlo Simulation

More detailed uncertainty analysis can be performed by Monte Carlo simulation. This technique, in principle, generates random values of stochastic input parameters according to their respective probabilistic characteristics. A large number of random parameter sets are generated to compute the corresponding model output. Then, analyses are performed upon the simulated model output to determine the statistical characteristics of the output such as the mean, variance, confidence interval and so on.

In the past, the main disadvantage often noted for Monte Carlo simulation is its computational intensiveness. Although this is still an issue for complex systems, it is increasing less of a problem due to the ready availability of powerful high speed personal computers. In fact, commercial softwares for Monte Carlo simulation are now available. One of these software named @RISK is used for uncertainty and reliability analyses in this project.

6 RELIABILITY IN STRUCTURAL FIRE DESIGN

6.1 INTRODUCTION

The lesser established a technical discipline is – mainly because it of being a relatively new one – the more uncertainties there are associated with the existing body of knowledge. This can be said of the field of fire engineering. The various uncertainties related to structural fire design have been described in chapter 4. The pertinent question here is, given all the uncertainties as described therein, how reliable is a structural fire design? Or, if the structure has been erected, what is the structural reliability of the building under the condition for which it has been designed – fire in this case? In more plain language, the question is, how stable is the building or what is the probability of collapse of the whole or part of the structure in the event that a fully developed broke out inside the building? To answer these questions, recourse has to be made to another relatively new discipline – reliability analysis.

6.2 THE CONCEPT OF RELIABILITY

In the broadest sense, reliability is associated with dependability, with successful operation, and with the absence of breakdowns or failures. However, in engineering analysis it is necessary to define reliability quantitatively in terms of probability. Thus reliability is defined as the probability that a system will perform its intended function for a specified period of time under a given set of conditions (Lewis 1994). "System" can refer to any product or processes like equipment, a building, sub-systems, components and parts.

A product or system is said to fail when it ceases to perform its intended function. When there is total cessation of function – an engine stops running, or a structure collapses – the system has clearly failed. Often, however, it is necessary to define failure quantitatively in order to take into account the more subtle forms of failure through deterioration or instability of function. Thus a motor that is no longer capable of delivering a specified torque or a structure that exceeds a specified deflection has failed. The choice of failure criteria is often arbitrary. So there is a need for consistency.

6.3 THE BASIC RELIABILITY PROBLEM

In structural fire design the problem of reliability may be cast as a problem of "capacity versus demand." Here, we are concerned that the fire resistance (capacity) of a building element (structural or otherwise) is sufficient to withstand the fire severity (demand) in a room. That is, the capacity must always be greater than the demand. The question of reliability arises because the determination of the available capacity (fire resistance) and the maximum demand (worst fire severity) are not simple or exact problems. Estimation and prediction are invariably necessary for these purposes; in the process, uncertainties are unavoidable for the simple reason that engineering information is incomplete. In order to explicitly represent or reflect the significance of uncertainty, the available capacity and demand may be modelled as random variables. In these terms, the reliability of the system may be realistically measured in term of probability. For this purpose, we define the following random variables:

R = Fire Resistance (Capacity)
S = Fire Severity (Demand)
M = Safety Margin defined as $M = R - S$

The equation $M = R - S$ is also known as the limit state function and can be denoted by $G(R,S)$. The objective of a reliability-based design is to ensure that given the outbreak of any fire the event ($M > 0$) or $G(R,S) > 0$ occurs throughout the useful or specified life of the building. This assurance is only possible in term of the probability $P(M > 0)$. This probability therefore represents a realistic measure of the reliability of the system. Conversely, the probability of the complimentary event ($R \leq S$) or ($M \leq 0$) is the corresponding measure of unreliability or failure of the design.

The probability of failure P_f is thus expressed as:

$$P_f = P(R - S \leq 0) = P(M \leq 0) \tag{6.1}$$

If the necessary probability density functions of R and S are available or can be approximated, that is $f_R(r)$ and $f_S(s)$ are known, and if R and S are continuous random variables which are statistically independent, the probability of failure P_f may then be expressed as follows:

$$P_f = P(R - S \leq 0) = \int_{-\infty}^{\infty} \int_{-\infty}^{s \geq r} f_R(r) f_S(s) dr ds \tag{6.2}$$

This is illustrated in fig.6.1, which shows the probability density functions (PDF) of R and S.

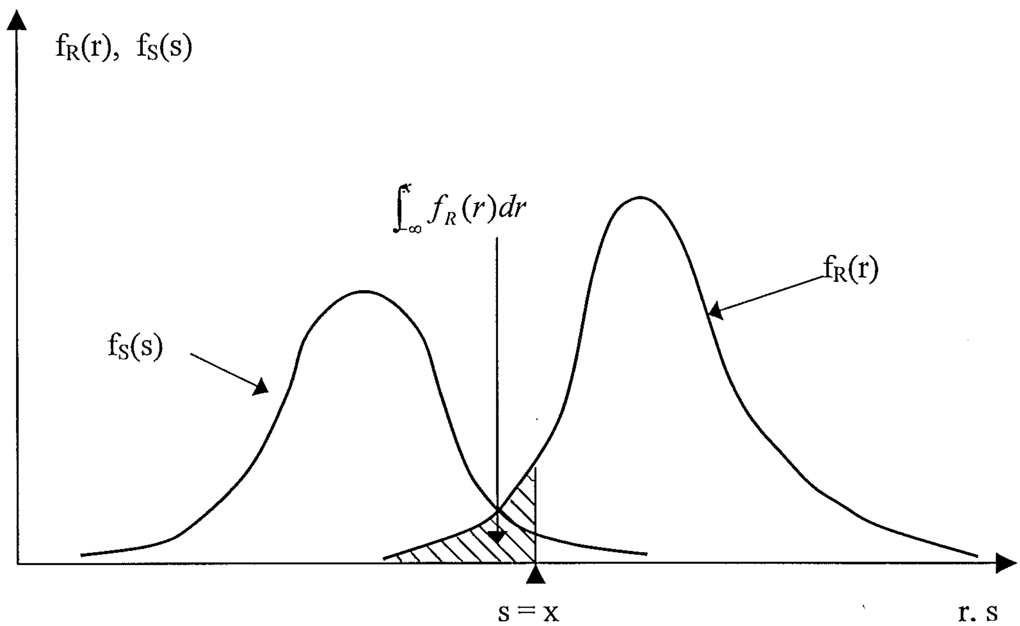


Figure 6-1 Basic Reliability Problem Represented by PDF of R and S

In statistical theory, for any random variable X, the cumulative distribution function (CDF), $F_X(x)$ is given by equation (6.3), provided that $x \geq y$. (Ang & Tang 1975)

$$F_X(x) = P(X \leq x) = \int_{-\infty}^{\infty} f_X(y)dy \tag{6.3}$$

Equation (6.2) can then be written in the form:

$$P_f = P(R - S \leq 0) = \int_{-\infty}^{\infty} F_R(x) f_S(x)dx \tag{6.4}$$

The integral in equation (6.4) is known as a "convolution integral" and is easier to solve than equation (6.2). The meaning of equation (6.4) can best be explained by reference to Fig.6.2.

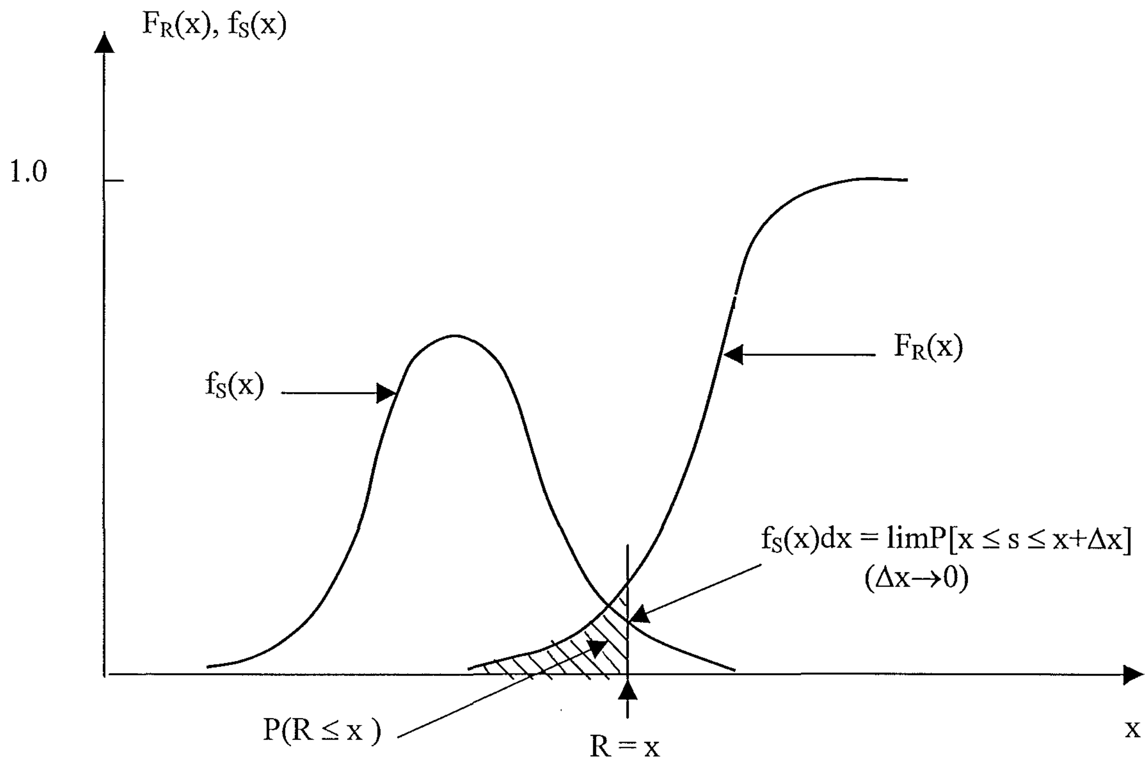


Fig. 6.2 Reliability Problem Represented by PDF of S and CDF of R

$F_R(x)$ is the probability that $R \leq x$, or the probability that the actual fire resistance R of the element is less than some value x. Let this represent failure. The term $f_S(x)dx$ represents the probability that the fire severity has a value between x and $x + \Delta x$ in the limit as $\Delta x \rightarrow 0$. By considering all possible values of x, that is, by taking the integral over all x, the total probability of failure is obtained.

The lower limit of integration shown in equation (6.2) and (6.4) may not be totally satisfactory, since a "negative" fire resistance is not usually possible. The lower limit of integration should strictly be zero, although this may be inconvenient and slightly inaccurate if R and S are modelled by distributions unlimited in the lower tail (such as the normal distribution). The inaccuracy arises strictly from the choice of distribution for R and S, and not from the theory involved with the equations (6.2) - (6.4).

6.3.1 Special Case: Normal Random Variables

When R and S are normal random variables with means μ_R and μ_S and variances σ_R^2 and σ_S^2 respectively, the random variable M is also normal with a mean and variance given by:

$$\mu_M = \mu_R - \mu_S \tag{6.5}$$

$$\sigma_M^2 = \sigma_R^2 + \sigma_S^2 \tag{6.6}$$

Equation (6.1) then becomes:

$$P_f = P(R - S \leq 0) = P(M \leq 0) = \Phi\left(\frac{0 - \mu_M}{\sigma_M}\right) \tag{6.7}$$

Where Φ is the standard normal distribution function (zero mean and unit variance). On substituting (6.5) and (6.6) into (6.7), the following expression is obtained:

$$P_f = \Phi\left[\frac{-(\mu_R - \mu_S)}{(\sigma_R^2 + \sigma_S^2)^{1/2}}\right] = \Phi(-\beta) \tag{6.8}$$

Where β is defined as the reliability index or safety index. From equation (6.8) it can be seen that as the difference between the mean of the fire resistance and the fire severity is reduced, the value of β decreases and P_f increases. Similarly, when either σ_R or σ_S or both are increased, β will decrease, with corresponding increase in P_f. The probability density of the random variable M, denoted by $f_M(m)$, and the "physical" meaning of β is illustrated in Fig.6.3.

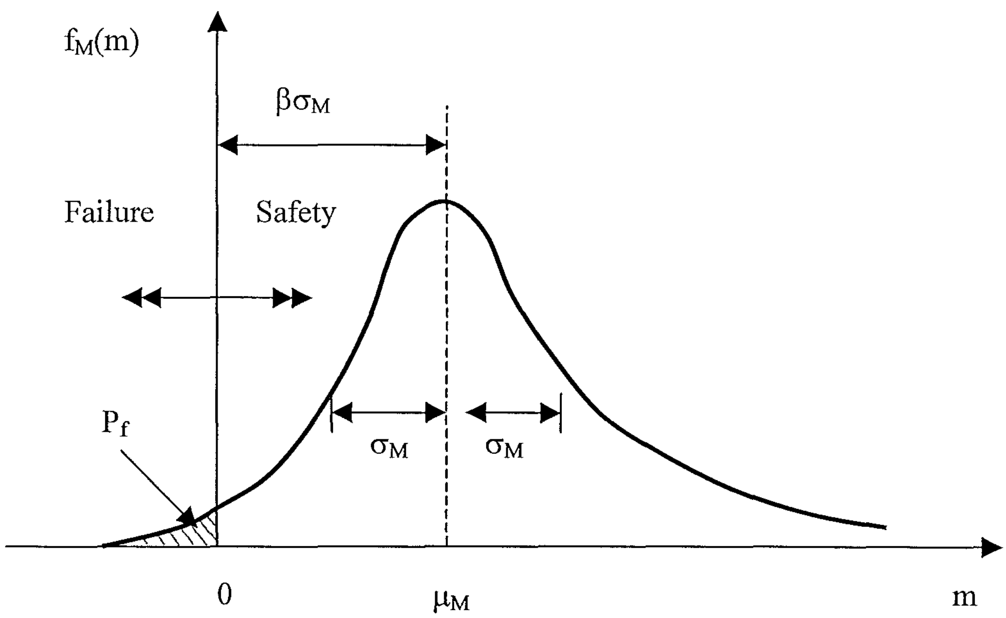


Fig. 6.3 Probability Density Function of the Safety Margin M

As shown in Fig.6.3, β is simply a measure (in units of standard deviation σ_M) of the distance that the mean μ_M is away from the origin m = 0. This point marks the boundary to the "failure region". Hence β is a direct measure of the safety of the structural element and greater β represents greater safety, or lower probability of failure.

6.4 METHODS OF RELIABILITY ANALYSIS

6.4.1 Preliminary

In the preceding sections, the basic reliability problem has been formulated and mathematical equations derived for calculating the total probability of failure. Thus, to determine the reliability of this "capacity-demand" problem, one only needs to evaluate the "convolution integral" shown in equation (6.4) to obtain the total probability of failure. The probability of non-failure or safe performance is a direct measure of reliability. However, in reality, the problem is generally not as simple as described earlier. First of all, closed form integration of equation (6.2) or (6.4) is only possible for some special cases, for example, when both R and S are normal random variables. In general, recourse must be made to numerical integration. Secondly, the simplified formulation of equation (6.4) is not sufficient for many real life problems. Usually several random variables will influence the capacity or resistance.

It follows that in general the resistance or capacity of a system is a vector function of various parameters. If the vector **X** represents the basic variables of the problem, then the limit state equation G(R,S) = R - S = 0 can be generalised as G(**X**) = 0. Consequently, equation (6.2) can be generalised as:

$$P_f = P[G(X) \le 0] = \int \ldots \int_{G(X)\le 0} f_X(x)dx \qquad (6.9)$$

Here, $f_X(x)$ is the joint probability density function for the n vector **X** of basic variables.

Equation (6.9) adds considerably to the complexity of the calculation of failure probability. While numerical integration may be possible for very small number (less than 5) of random variables, the problem is considered not feasible to be solved analytically even on computers (Stewart & Melchers 1997). However, techniques have been developed to handle these problems. Two broad classes of the more commonly used techniques are the First Order Second Moment methods and the Monte Carlo Simulation method. These are briefly described below and their suitability for assessing reliability of structural fire design would be discussed.

6.4.2 First Order Second Moment Method

In the First Order Second Moment (FOSM) method the terminology 'second moment' refers to the description of all the random variables in terms only of their mean (the 'first moment') and their variance (the 'second moment'). The normal distribution is adequately specified by these two 'moments' and for simplicity and applications one could read 'second moment' methods simply as performing calculations only with normal distributions. It does not mean that the method does not apply to non-normal distributions – its application merely implies that whatever the distribution attached to a random variable, only the first two moments are considered are considered for calculation. Higher moments, which might describe the skew and flatness of the distribution are ignored.

The term "first order" defines the expression for the failure condition or limit state function as a linear function. In section 6.3 above, the limit state function G(R,S) was given by M = R - S which is clearly linear. The probability of failure P_f is given by;

$$P_f = \Phi(-\beta), \qquad \beta = \mu_M/\sigma_M \tag{6.10}$$

Where β is the reliability index or safety index and Φ is the standard normal distribution. P_f, as defined above, is the exact probability of failure if both R and S are normally distributed. For other distributions of R and S, both the random variables and the original problem must be transformed to the standard normal space - a process which requires, in the general case, the use of the Rosenblatt (1952) transformation. Then the P_f determined as defined in (6.10) is only the nominal failure probability. Consequently, the conventional practice for such cases is not to refer to failure probability at all, but to β, the safety index (Melchers,1987).

A further complication arises when there are more than two random variables or where the limit state function is a composite of several components such as denoted by equation (6.9). Also, the limit state function is likely to be non-linear. In the FOSM method the non-linear function, say G(**X**), is linearised through a first order Taylor's series expansion about an appropriately chosen point, say **x***. By truncating the series at linear terms, the approximate first order mean (μ_x) and variance (σ_x^2), in form of algebraic expression, are then obtained. The expression for the safety index is then given by:

$$\beta_{x*} = \frac{\mu_x}{\sigma_x}$$

The subscript for β means that its value depends on the choice of the expansion point **x***.

The foregoing is just a very brief introduction to the FOSM method of reliability analysis. It has to be appreciated that usage of this method entails:

- an accurate formulation of the limit state function,
- transformation (e.g by Rosenblatt transformation), if necessary, to equivalent normal distribution,
- linearisation of non-linear function by Taylor series expansion about an appropriate point.
- application of an iterative procedure or an algorithm in a computer software in order to arrive at a most accurate value of the safety index β.

FOSM methods are extensively used in reliability analysis of various engineering systems, including civil engineering structure at normal temperature. The main reason for the wide application of FOSM method in these more established areas is the availability of properly formulated limit state functions. This method is also applicable in fire engineering to assess the reliability of different aspects of fire safety design, including structural fire design. However, these methods have the following disadvantages:

- the 'design' or 'checking' point must be accurately identified for each limit state function, in order to obtain a sensible value of β,

- non-linear limit state functions are not easily handled and may give rise to inaccuracies,
- the transformation of non-normal distributions to equivalent normal distributions is very difficult, especially if the random variables are correlated.

A more suitable method for assessing the reliability of structural fire design is described in the next section.

6.4.3 The Method of Monte Carlo Simulation

Monte Carlo simulation techniques involve "sampling" at "random" to simulate artificially a large number of experiments and to observe the results. In the case of analysis for any structural reliability, with problem type as defined by equation (6.9), this means sampling each random variable X_i randomly to give a sample value x_i. The limit state function $G(x) \leq 0$ is then checked. If the limit state function is violated, that is the structure or structural element has "failed", this event is counted and recorded. The experiment is repeated many times, each time with a randomly chosen vector **x** of x_i value. If N trials are conducted, the probability of failure is given approximately by:

$$P_f = \frac{n(G \leq 0)}{N}$$

Where $n(G \leq 0)$ is the number of trials for which $G \leq 0$ or failure occurred.

Obviously the number N of trials required is related to the desired accuracy for P_f. In other words, in the Monte Carlo method a game of chance is constructed from known probabilistic properties in order to solve the problem many times over, and from that to deduce the required result (e.g. the failure probability).

To apply Monte Carlo techniques to practical problems the following broad outline of procedures are required (Melchers, 1987):

- to develop systematic methods for random and numerical "sampling" of the basic variable **X**;
- to select an appropriate economical and reliable simulation technique;

- to consider the effect of the complexity of calculating G(**X**) and the number of basic variables on the simulation technique used;
- for a given simulation technique to be able to determine the amount of "sampling" required to obtain a reasonable estimate of P_f.
- to account for correlation or dependence between all or some of the random variables in the model

The above stated procedures are for setting up a model for Monte Carlo simulation analytically, and the mathematics involved can be as difficult as that for the FOSM method. After setting up the model, the actual multiple runs have to be done on a computer in order to obtain more accurate results. In the past, lack of computer capability had limited the use of Monte Carlo simulation (Stewart & Melchers 1997). This is not an issue now. In fact several computer software are now commercially available to enable Monte Carlo simulation to be carried out on any reasonably high speed and capacity personal computer. Use of commercial software for Monte Carlo simulations has greatly enhanced quantified risk assessment and has the following advantages (Vose, 1996):

- the distributions of the model's variables do not have to be approximated in any way (items can be selected from the software's menu)
- correlation and other inter-dependencies can be modelled
- the level of mathematics required to use Monte Carlo simulation software is quite basic (compared to the analytical approach)
- greater level of precision can be achieved by simply increasing the number of iterations on the computer
- changes to the model can be made very quickly and the results compared with the previous models.
- The computer does all of the work required in determining the probability distribution function of any desired outcome.
- Monte Carlo simulation is widely recognised as a valid technique so its results are more likely to be accepted.

A number of software package are commercially available to carry out risk analysis in general and some of those that are add-ins to spreadsheet suitable for setting up a model for Monte Carlo simulation are as follows:

(i) @RISK Software

@RISK software (Palisade 1995), originally developed for use with Lotus 1-2-3 is also available for the Excel spreadsheet. It has very sophisticated set of features but remains easy to use.

(ii) Crystal Ball

Crystal Ball (Decisioneering 199?) has a lot of features in common with @RISK but lacks some its sophistication.

(iii) Predict!

Predict! (Risk Decision199?) is a stand-alone product with its own spreadsheet. It uses a different format for its formulae and lacks a lot of the sophistication of modern spreadsheet that @RISK and Crystal Ball capitalise on.

@RISK will be the software used to carry out reliability analysis in the two case studies described in chapter 7 and 8 of this report. A brief description of the @RISK software follows in the next section.

6.4.4 Description of @RISK Software

@RISK software is an "add-in" to Microsoft Excel or Lotus 1-2-3 for window spreadsheet programs. This software brings advanced modelling and risk analysis capability to these two spreadsheet program.

@RISK uses the technique of Monte Carlo simulation for risk analysis. With this technique, uncertain input values in the spreadsheet are specified as probability distributions. There are more than 30 probability distributions to choose from @RISK's menu - ranging from the Beta to the Weibull distribution. Distribution functions can be added to any number of cells and formula throughout the worksheets and can include arguments which are cell references and expressions, thus allowing extremely sophisticated specification of uncertainty. Other options available for controlling and executing a simulation in @RISK are:

- Latin Hypercube or Monte Carlo sampling
- Any number of iterations per simulation
- Any number of simulations in a single analysis
- Continuing a simulation after viewing results and performing more iterations if necessary
- Seeding the random number generator

@RISK graphs an approximation to probability distribution of possible results for each output cell selected in the spreadsheet. Other graphics include:

- Relative frequency distributions and cumulative probability curves
- Summary graphs for multiple distributions across cell ranges (a worksheet row or column)
- Statistical reports on generated distributions
- Probability of occurrence for target values in a distribution
- Export of graphics as window metafiles for further enhancement.

Risk analysis in @RISK is a quantitative method that seeks to determine outcomes of a decision situation as a probability distribution. In general, techniques in an @RISK risk analysis encompass four steps:

(i) Developing a model - by defining the problem or situation in Excel or Lotus 1-2-3 format

(ii) Identifying Uncertainties - of variables in the work sheets and specifying their possible values with probability distributions, and identifying the uncertain work sheet results that require analysis.

(iii) Analysing the Model with Simulation - to determine the range and probabilities of all possible outcomes for the results of the work sheets

(iv) Making a Decision - based on the results provided and personal preferences

@RISK helps with the first three steps, by providing a powerful and flexible tool that works with Excel or 1-2-3 to facilitate model building and risk analysis. The results that @RISK generates can then be used by the decision maker to choose a course of action.

For a more comprehensive description of the @RISK program, readers are advised to consult the @RISK users annual (Palisade 1995)

7 CASE STUDY I: RELIABILITY ANALYSIS OF PERFORMANCE OF CONCRETE SLAB

7.1 Introduction

In structural fire design, critical building components like structural beams, columns, walls, floors and doors are designed to have fire resistance rating. The design consideration is that the fire resistance rating should be more than sufficient to withstand the worst fire severity expected in the building. The methodologies for quantifying both the fire resistance and fire severity have been explained in chapter 2. If a building had been designed for fire safety the "design fire resistance" of a building component is normally calculated based on the "design fire severity" in a fire compartment. But in many cases building components are not designed with fire resistance as the primary concern. However, this does not mean that such components are not fire resistant at all. The fire resistance of these building components can still be assessed by calculation or by applying empirical correlations, as explained in section 2.2

The objective of this case study is to assess the reliability of applying empirical correlations for determining the fire resistance of a building component and the fire severity, given the outbreak of a fully developed fire in the room. The building element selected for assessment is a monolithic concrete slab.

7.2 Structural Performance of a Monolithic Concrete Slab Under Fire Condition – Deterministic Calculation

7.2.1 Fire Compartment Data

The structural performance under fire condition of a monolithic concrete slab, which forms the floor of a compartment, would be assessed here. Data related to the fire compartment are as follows:

Room depth: 7m

Room width: 5m

Room height: 2.5m

Height of window: 1.5m

Width of window: 3.5m

Internal surface lining material: All concrete construction

Fuel load density: $800MJ/m^2$ [Hazard category 2 (BIA 1995)]

7.2.2 Fire Resistance Rating of slab

The fire resistance of dry monolithic normal weight concrete slabs based on obtaining a failure temperature rise of 250°F at the unexposed surface is given by the following semi-empirical formula (Harmathy 1970, Allen and Harmathy 1972):

$$R_1 = 0.205 \frac{(\rho c)^{1.2} L^{1.85}}{k^{0.65}} \quad (7.1)$$

Where :

R_1 = the fire resistance of slab based on heat transmission criterion (hour)

L = thickness of slab (ft)

ρ = density of concrete (lb/ft^3)

c = specific heat of concrete (Btu/lb°F)

k = thermal conductivity of concrete (Btu/ft h°F).

If no data on the thermal properties of the concrete are available, the following conservative values may be used for these properties:

k = 1.0 Btu/ft h°F (1.73/mK) for normal weight concrete

k = 0.45 Btu/ft h°F (0.78W/mK) for lightweight concrete, and

c = 0.20 Btu/lb°F (837.4J/kgK) for both concretes.

In this case equation (7.1) becomes, for normal weight concrete:

$$R_1 = 0.03\rho^{1.2} L^{1.85} \quad (7.2)$$

In term of minutes, equation (7.2) can be expressed as follows:

$$F = 1.8\ \rho^{1.2} L^{1.85} \tag{7.3}$$

Where: F = Fire Resistance Rating (minutes)

7.2.3 Fire Severity

The Eurocode 1 formula (EC1 1994) for the time equivalence, t_e , for fire severity in a room is given by:

$$t_e = k_b\, w\, e_f \quad \text{(min)} \tag{7.4}$$

where:

e_f is the fuel load (MJ/m^2 of floor area)

k_b is a parameter to account for different compartment linings

w is the ventilation factor given by

$$w = \left(\frac{6.0}{H_r}\right)^{0.3} \left[0.62 + \frac{90(0.4-\alpha_v)^4}{1+b_v\alpha_h}\right] > 0.5$$

where: H_r is the compartment height (m)

$\alpha_v = A_v/A_f \qquad 0.05 \le \alpha_v \le 0.25$

$\alpha_h = A_h/A_f \qquad \alpha_h \le 0.20$

$b_v = 12.5(1+10\alpha_v - \alpha_c^2)$

A_f is the floor area of the compartment (m^2)

A_v is the area of vertical openings (m^2)

A_h is the area of horizontal openings (m^2)

7.2.4 Limit State Equation

The limit state equation for the calculation is thus given by :

$$M = F - t_e$$

Where M is the safety margin (min)

That is, $M = 1.8\rho^{1.2}L^{1.85} - k_b w\, e_f$

7.2.5 Performance of slab

For a concrete slab thickness of 0.33ft (4 in.) and with normal weight concrete density taken as 132 lb/ft^3 (2200kg/m^3), the fire resistance is given by:

$$F = 1.8(132)^{1.2}(0.33)^{1.85} \text{ min}$$
$$= 81 \text{ min.}$$

Based on the data from section 7.2.1, the following are obtained:

Ventilation factor w = 1.26.

With concrete as the room internal surface lining material, the thermal inertia

$$= \sqrt{(k\rho c)}$$
$$= \sqrt{(1.73 \times 2200 \times 837.4)}$$
$$= 1785 \text{ J/m}^2\text{Ks}^{0.5}$$

$\therefore$ Conversion factor $k_b = 0.055$ (Buchanan 1994)

The fire severity $t_e = k_b\, w\, e_f$

$$= (0.055)(1.26)(800)$$
$$= 55 \text{ min.}$$

The safety margin $M = F - t_e$

$$= 81 - 55$$
$$= 26 \text{ min.}$$

The above deterministic calculation shows that, with a safety margin of 26 minutes, the concrete slab's fire resistance rating is more than sufficient to withstand the expected fire severity in the room.

7.3 Structural Reliability of Monolithic Concrete Slab Under Fire Condition

The deterministic calculation of the safety margin has been carried out using a single value of the parameters involved. To assess the structural reliability of the slab under fire condition, variation in the values of all the parameters has to be accounted for. This can be done by carrying out Monte Carlo simulation on the @RISK software. The parameters for simulations are treated as random variables which take on a range of value defined by their respective probability distribution functions. The characterisation of the parameters as random variables are described below.

(a) Room Depth

Variation is mainly due to construction and measurement errors. These errors are estimated to be in the order of 1%. The room depth is most realistically treated as a random variable which exhibits a triangular distribution. Triangular distribution is characterised by three parameters viz. the most likely value, a minimum and a maximum value. Both the minimum and maximum values have zero probability of occurring. The nominal depth of 7.0m can be regarded as the most likely value. The addition and subtraction of 1% of the most likely value give the maximum value of 7.07m and a minimum value of 6.93m respectively.

(b) Room Width

Similarly, the room width is treated as a random variable with triangular distribution. The most likely value is nominal width of 5.0m. The minimum value is 4.95 and the maximum value is 5.05m

(c) Room Height

In the same manner, the room height exhibits a triangular distribution with a most likely value of 2.50m, a minimum value of 2.47m and a maximum of 2.53m

(d) Height of Window

This is assumed to exhibit triangular distribution with a minimum value of 1.48m, a most likely value of 1.50m and a maximum value of 1.52m.

(e) Width of Window

This is assumed to exhibit triangular distribution with a minimum value of 3.46m, a most likely value of 3.5m and a maximum value of 3.54m.

(f) Fuel Load Density

A hazard category of 2 is assumed for this room and a fuel load density of 800MJ/m^2 of floor area is used in the deterministic spreadsheet calculation. This is the design value recommended in the Annex to Fire Safety Documents of the Approved Documents (BIA 1995b) for fire hazard category 2. However, this design value is the 80^{th} percentile fire load of the range of fire load density stated for the relevant fire hazard category. The variation in the fuel load density can be assumed to be normally distributed with a coefficient of variation of 25% (CIB 1986). As such, the mean value of the fuel load density is worked to be 660MJ/$m^{2.}$ The standard deviation is therefore 165MJ/m^2.

(g) Compartment Lining Factor k_b

The value of k_b depends on the value of the thermal inertia [$\sqrt{(k\rho c)}$] which, for concrete, can range from a minimum value of around 420 J/$m^2Ks^{0.5}$ to a maximum value of around 2050 J/$m^2Ks^{0.5}$. The value of k_b can then range from around 0.055 to 0.07min.$m^{2.3}$/MJ. The random variable k_b is assumed to be normally distributed with a mean of 0.060 and a coefficient of variation of 0.10. The distribution is truncated with a minimum value of 0.055 and the maximum value of 0.070 min.$m^{2.3}$/MJ

(h) Density of Concrete

Density of concrete varies widely, depending on the aggregate mix, and can range from 120 lb/ft^3 (1900kg/m^3) to 157 lb/ft^3 (2500kg/m^3). A truncated normal distribution between these lower and upper values with a mean of 132 lb/ft^3 (2200kg/$m^{3)}$ and standard deviation of 7 lb/ft^3 (110kg/m^3) is assumed.

(i) Thickness of Concrete Slab

The concrete slab thickness is expected to be uniform with any variation due to the casting process and shrinkage. The nominal thickness of the concrete slab is 0.3333ft (4in), which can be regarded as the mean value of the assumed normal distribution of the slab thickness. The coefficient of variation is small and assumed to be 0.02. The truncated normal distribution is assumed to have a minimum value of 0.3133ft and a maximum value of 0.3533ft. The standard deviation is 0.0067ft..

7.3.1 Results of Simulation

The Monte Carlo simulation using @RISK programme would generate relative frequency distributions of several outputs. The outputs' frequency distributions of interest are that for the safety margin, the fire resistance of the concrete slab and the fire severity in the room. The relative frequency distributions for the fire resistance and fire severity is shown in Fig.7.1

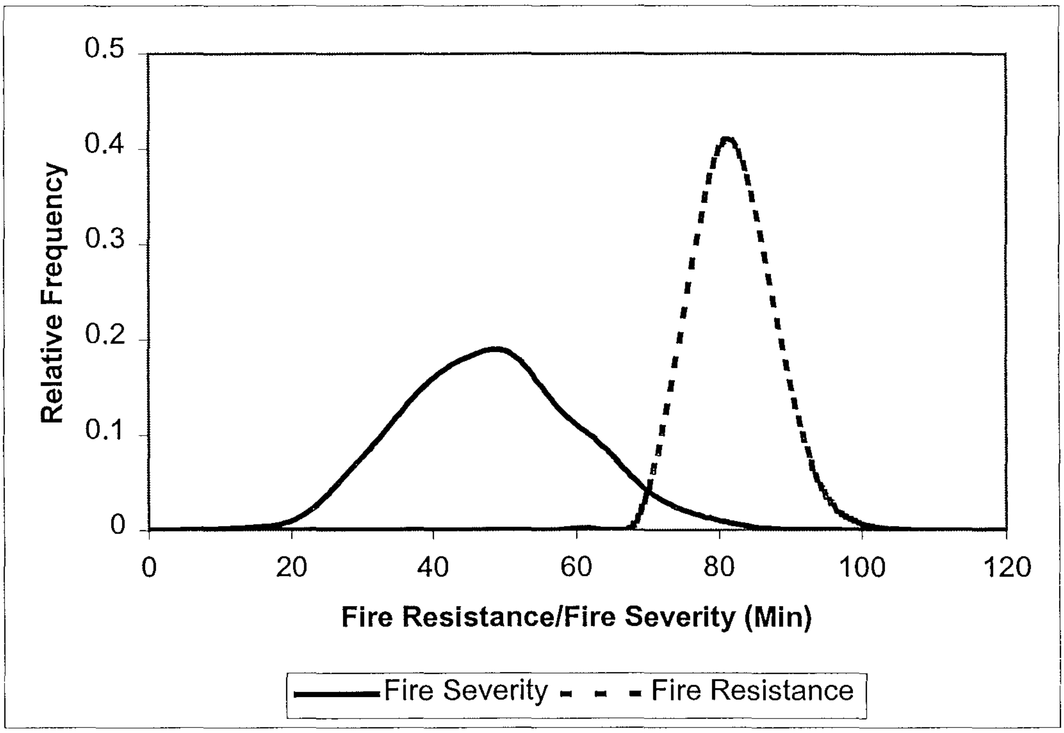


Figure 7-1Frequency Distribution of Slab's Fire Resistance and Room Fire Severity

Figure 7-1 very clearly illustrates the situation as that of a capacity demand problem. However, not much useful computation can be made from figure 7-1.

In assessing the structural reliability of the concrete slab under fire condition, the output of interest is the probability of failure of the slab, that is, the probability of the value of the safety margin being less than or equal to zero. The probability of failure can be easily obtained from the cumulative distribution of the safety margin. @RISK has provision for reformatting frequency distribution into cumulative distribution. The cumulative distribution for the safety margin is shown in figure.7.2.

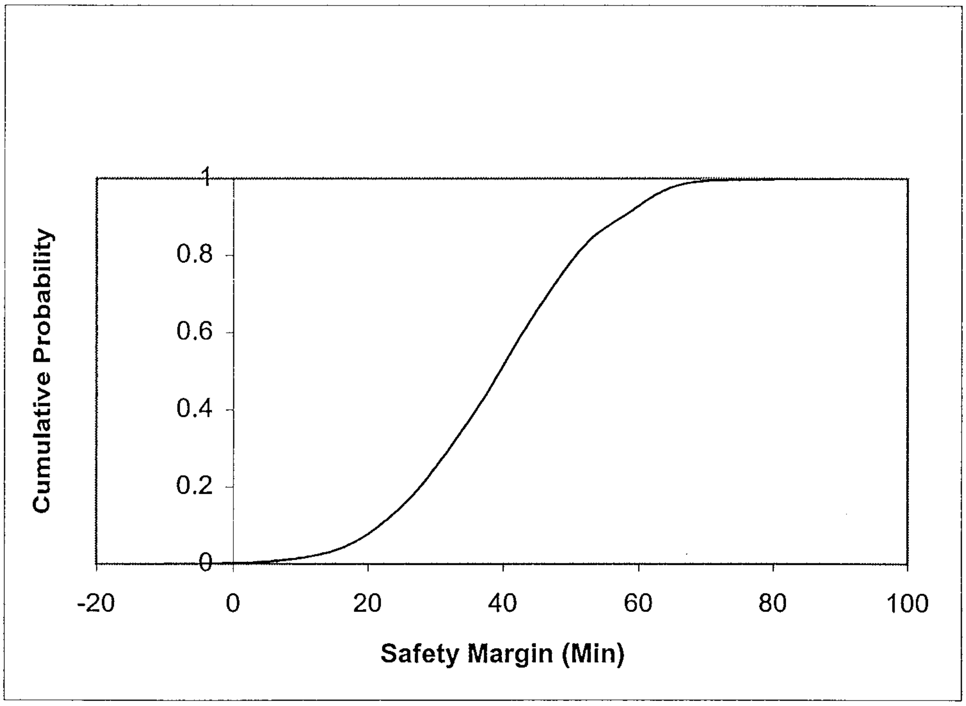


Figure 7-2 Cumulative Distribution of the Safety Margin

In figure 7-2 the probability of failure, P_f, is the value of the point where the curve intersects the y-axis. In this particular case, the scale is too small for the value of intersect to be read off. Recourse has to be made to the data generated by @RISK which can be exported to the software's spreadsheet. Extract of the data are shown in Table 7-1.

Table 7-1Extract from Spreadsheet Data for CDF of Safety Margin

M	Prob	-6	2.03E-04	-1.5	1.28E-03
-9.75	0	-5.5	2.90E-04	-1	1.47E-03
-9.5	0	-5	3.81E-04	-0.5	1.68E-03
-9	0	-4.5	4.79E-04	0	1.91E-03
-8.5	0	-4	5.84E-04	0.5	2.18E-03
-8	0	-3.5	6.99E-04	1	2.47E-03
-7.5	0	-3	8.25E-04	1.5	2.81E-03
-7	4.00E-05	-2.5	9.62E-04		
-6.5	1.21E-04	-2	1.11E-03		

From Table 7-1 the probability of failure of the concrete slab is 0.00191 or around 0.2%. Assuming that the distribution of the safety margin is normal, this probability of failure corresponds to a value of the reliability index , β, of 2.89.

7.4 DISCUSSION

This case study very clearly illustrates a situation whereby what appears to be a very safe design from deterministic calculation is not necessarily so when the reliability of the performance is assessed. On the other hand, the existence of even a small probability of failure does not mean that the calculation is not valid. What is more important is the question of what constitutes an acceptable probability of failure or what is the acceptable minimum reliability index? The answers to these questions should best be obtained from the national codes or regulatory documents.

However, it should be mentioned that whilst most national codes do specify the acceptable reliability index for the ultimate limit state due to normal gravity load, wind load, seismic load, and combination of loads, the same cannot be said for loading during fire. Be that as it may, it should also be appreciated that the acceptable probability of failure is normally calculated to be almost the absolute probability in the sense that almost all the uncertainties have to be accounted for. Thus, the main reason why acceptable probability of failure has not been specified for structural reliability under fire condition is that there may exist too many uncertainties in most fire situations, which cannot be easily accounted for.

In this case study, the probability of failure of slab is conditional upon the outbreak of a fully developed fire in the room. Of interest then is what is the probability of a fire becoming fully developed given certain probability of ignition or probability of flashover. Then there is probability of active protection system like sprinkler activating or the probability of successful intervention by the fire service to consider. The list can go on and if most of the events are independent, the near absolute or overall probability of failure of the concrete slab in fire situation, by applying the multiplication rule, would be reduced to practically zero.

This does not mean that the conditional probability of failure is not useful. A better term for this conditional probability is "nominal" probability (Melchers 1987). Nominal probability of failure is a very useful to indicate flaws or deficiency in a design or an existing building. In fact nominal measures of failure probability can be used as substitutes for more accurately determined measures if the effects of human error in particular are assumed to be similar for similar situations or structural components (Melchers 1987).

An attempt at calculating the acceptable probability of failure for structural fire design is described in chapter 10. The reliability of performance of the concrete slab will be further assessed in that chapter.

8 FIRE ENGINEERING DESIGN OF A STEEL FRAMED HOTEL BUILDING

8.1 PRELIMINARY

In this chapter, a detailed description is given of a structural fire engineering design of a steel framed hotel building. The original fire engineering design was done by an engineering consultant firm with expertise in fire resistance of steel structures. To preserve confidentiality, the name of the designer, building owner and building name will not be referred to in this report. As the original design report could not possibly show all data or calculations used in the design process, a close approximation was made of the original design and calculations to show results which are as near as possible to the original results.

8.1 OVERVIEW OF ORIGINAL DESIGN

8.1.1 Building Description

The fire engineering design was actually carried out on an extension to an existing hotel. The extension included a few additional floors of more than 100 rooms constructed on top of the existing building. The new structure is framed entirely in structural steel, with steel beams acting compositely with a concrete floor slab on profiled steel decking. Columns are all steel I section and the lateral load resisting system consists of both eccentrically-braced and moment-resisting frames.

8.1.2 Design Philosophy

The fire engineering design dealt only with the requirements for adequate fire resistance of the steel framed structure, which had been designed by an architectural and structural engineering consultant firm. Fire engineering analyses was employed to predict the behavior of natural or realistic fire in representative rooms and the structural responses of the structural elements in these rooms. The goal was to ascertain whether passive fire protections should be applied to the structural elements in order to achieve the required fire resistance or period of structural stability required by regulation. The whole analyses were performed on the premise that active fire protection system like the sprinkler did not operate and there was no intervention by the fire services.

8.1.3 Methods of Analysis and Results

Based on the Eurocode I (EC1 1994) relationship for "real" fire, the air time temperature curves for a range of realistic natural fires in different fire compartments are obtained. These are used to calculate the maximum temperatures attained by the unprotected steel members, which are then compared with the limiting steel temperatures assessed in accordance with NZS 3404 Steel Structures Standard (SNZ 1997). The steel members are shielded from the direct effect of fire by the presence of a suspended "Gib-grid" ceiling system or Gib-board wall claddings. If the maximum steel temperature in a fire does not exceed the limiting steel temperature, then the beam or column continues to support the design loads throughout and after the fire.

The fire engineering analyses show that the maximum temperatures that the steel members may reach in a "real" fire are less than the steel limiting temperature.

The conclusion of the fire engineering design is that the steel beams and columns would continue to support their long-term design loads throughout and after a fire and therefore do not need any passive fire protection to meet the performance requirement of the New Zealand Building Code or any realistic requirements of the owner for satisfactory protection of the building and contents.

8.2 PRESENT FIRE ENGINEERING ASSESSMENT

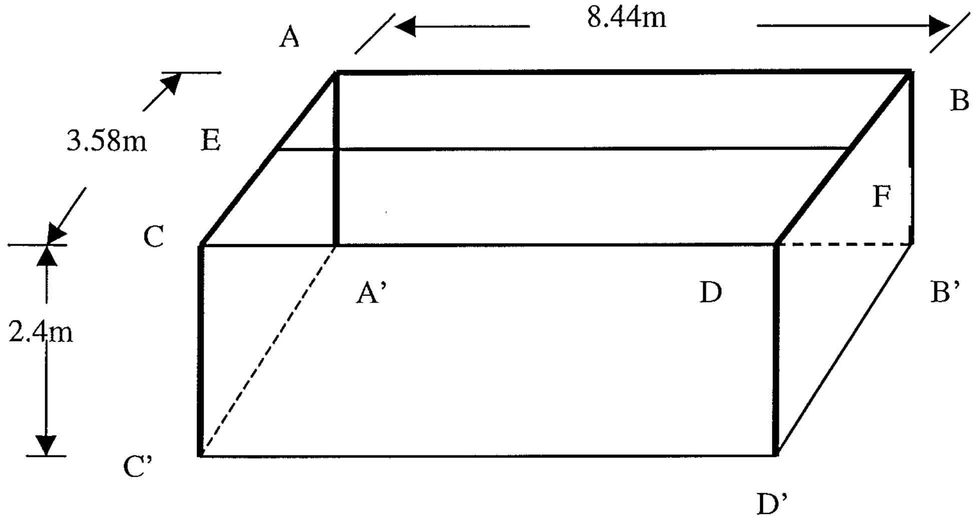
8.2.1 Overview of Methodology

The structural design process for fire resistance requires verification that the provided fire resistance exceeds the design fire severity. Verification may be in the time domain, the temperature domain or the strength domain. The time and the temperature domain will both be used in this project so that accuracy of both can be compared. To carry out this verification it is first necessary to set up the fire model, heat transfer model and the structural response models.

In the same manner as the original design, time-temperature curves for parametric fires in any room can be worked out on spreadsheets. These curves are used in spreadsheets to calculate the maximum steel temperatures attained by the beams or columns in the room. The spreadsheet calculation is based on the method by Gamble (1989) and Milke and Hill (1996). The structural steel members are not directly protected by sprayed on or encased protective material. The only form of protection is the presence of a monolithic 'Gib-grid' ceiling system using 12.5mm standard Gib-board which shields the steel beams and the 9.5mm standard Gib-board claddings which shield the steel columns. These ceilings and wall claddings were all conservatively assumed to fail 20 minutes after flashover and thence to fully expose the steel members to the hot gases. The time to failure of 20 minutes for suspended ceiling and wall cladding are based on research carried out by BHP Research Melbourne Laboratories (Thomas et al 1993). With the ceiling and wall claddings in place, the steel members are treated as lightly insulated members and to be treated as unprotected when the insulation failed. Models are also set up for the case where the unprotected steel members, without the benefits of the shielding effect from the ceiling and wall claddings, are exposed to the fire and the case where the members are protected by sprayed on mineral fibre. The shielding effects of the ceiling and wall claddings are also ignored in the latter case. The detailed modelling and assessment are described below.

8.2.2 Room and Steel Members' Dimensions

In this design of an extension to a hotel, all four storeys of the extension consist of bedrooms and suites. No new kitchen, bar or function room are included. Dimensions of rooms, window and door openings and sizes of steel beams and columns are obtained from the architectural and structural drawings. A representative room, which is a corner room, is selected as a fire compartment for assessment. An isometric view of the fire compartment showing the structural steel members in the compartment is shown in Fig. 8.1.



Beam Size		Column Size	
AB	460 UB 74.6	AA'	310 UC 158
EF/CD	310 UB 40.4	BB'	310 UC 158
AC	460 UB 74.6	CC'	310 UC 137
BD	460 UB 74.6	DD'	310 UC 137

Figure 8-1 Isometric View of Fire Compartment showing the Steel Members

Before carrying out fire engineering assessment for structural stability under fire situation it is necessary to assess the structural members' performance under ambient condition. In this case, ambient condition means the building is subject only to gravity loading at normal working temperature. No account is made for wind, seismic and snow loading. This is considered justified because under fire situation the probability of other extreme loads occurring together with gravity load is very low (Turkstra 1970).

8.2.3 Structural Performance under Ambient Condition

8.2.3.1 Design Action on Steel Members

The steel members are shown in Fig.8-1. For the steel beams the design action is the maximum bending moment resulting from the combination of loads. The design action on steel columns is the compressive load imposed on the columns. These steel members have been designed for combination of gravity, wind and seismic loads. In assessing their performance under both ambient and fire conditions, only gravity loads are considered.

8.2.3.2 Design Gravity Loading

With reference to Fig.8.1 the following data are used for load calculation:

Floor: 8.44m depth; 3.58m width; 0.130m concrete slab thickness

Tributary area, A_t, for supporting beam = 8.44 x 3.58/2 = 15.11m^2

Density of concrete = 2200kg/m^3

The dead load, G, due to the weight of concrete slab, weight of walls and self weight of steel beams are worked out below:

Weight of concrete slab = 0.130 x 22 kPa = 2.86kPa

Weight of partitions, etc = 0.50 kPa

Self weight of 310UB 40 beam = 0.40 kN/m

= 0.40 /(3.58/2) kPa

= 0.22 kPa

Therefore, dead load, Q = 2.86 + 0.50 +0.22 = 3.58 kPa

From NZS4203 (SNZ 1992), the basic live load, Q_b, is taken to be 2 kPa. (for bedroom-institutional).

Area reduction factor, ψ_a, is given by $1.0 \geq \psi_a = 0.4 + 2.7/\sqrt{A_t}$

that is, $\psi_a = 0.4 + 2.7/\sqrt{15.11} = 1.09$

Therefore $\psi_a = 1.0$

The live load, $Q = \psi_a Q_b$, is therefore 2.0 kPa.

The load combination for the ultimate limit state is given by:

$$L_u = 1.2G + 1.6Q \tag{8.1}$$

Therefore, $L_u = 1.2(3.58) + 1.6(2.0) = 7.50$ kPa

8.2.3.3 Performance of Beam

The combined load per unit length of beam, w, = 7.50 x 3.58/2 kN/m
= 13.43 kN/m

To be conservative, the steel beam is assumed to be simply supported. The bending moment at the centre of the beam is given by (Benham, et al 1996):

$$M = \frac{wL^2}{8} \tag{8.2}$$

Therefore $M = 13.43(8.44)^2/8 = 119.58$ kNm

Assuming that the structural steel is made from grade 300 steel, with a yield stress σ_Y of 320MPa, the required plastic section modulus, Z_p, for the case of plastic collapse of the beam, with the formation of a single plastic hinge at the centre is given by:

$$Z_P = \frac{M}{\sigma_Y} \tag{8.3}$$

Therefore, the required plastic section modulus, $Z_p = 119.58/320 \times 10^3 \text{ m}^3$
$= 373.7 \times 10^3 \text{ mm}^3$

The 310 UB 40.4 beam has a nominal plastic modulus of 633 x 10^3 mm^3 about the principal x axis. Applying a strength reduction factor, $\phi = 0.9$, the beam capacity is 570 x 10^3 mm^3 which is adequate. Similar analysis is carried out for other beam in the room and the results tabulated in Table 8.1.

This analysis ignores any composite action between the steel beam and the concrete slab.

Table 8-1Performance of Beams under Normal Condition

Steel Beam & Section	Required Z_p (10^3 mm^3)	Provided ϕZ_e (10^3mm^3)	Remark
EF: 310 UB 40.4	373.7	570	ok
AB: 460 UB 74.6	366	1494	ok
AC: 460 UB 74.6	317	1494	ok
BD: 460 UB 74.6	159	1494	ok

8.2.3.4 Performance of Columns

The room shown in Fig. 8-1 is one of several rooms of same design in one of the floors of the 4 storeys extension to an existing building. As indicated in the architectural drawings, the same column sizes are used throughout the 4 storeys. In this analysis, columns on the lowest floor are considered so that the imposed load is a total from the three storeys above and the roof. The dead load imposed by the roof is assumed to be half that imposed by one floor.

Interior Gravity Column (Column CC')

Tributary area (exluding roof), A_t, = 3{8.44 x 2.5(3.58)} m^2

$=227 \text{ m}^2$

Area reduction factor, ψ_a, $= 0.4 + 2.7/\sqrt{A_t}$

$= 0.4 + 2.7/\sqrt{227}$

$= 0.58$

Roof Data

Tributary area $= 8.44 \times 2.5(3.58)$

$= 75.5 \text{ m}^2$

$\psi_a = 0.4 + 2.7/\sqrt{75.5}$

$= 0.71$

From Table 3.4.1 of NZS 4203:1992 the basic live load, Q_b, is taken as 0.25 kPa

Loading on Column

Design gravity load on interior column, CC', N^*_g, is calculated as:

$$N^*_g = [1.2x3.58 + 1.6 \times 0.58 \times 2.0]227 + [1.2 \times 0.5 \times 3.58 + 1.6 \times 0.71 \times 0.25]75.5$$

$$= 1580 \text{ kN}$$

Using design criteria :

$$\phi N_{cy} \geq N_g^* \qquad (8.4)$$

where N_{cy} is the nominal member capacity for buckling about minor principal y axis, the following result is obtained:

$$\phi N_{cy} \geq 1580$$

or, taking $\phi = 0.9$, $N_{cy} = 1756$ kN

Actual column size: 310 UC 158

- Actual length, L, = 4.5m
- End restraint condition: One end translation and rotation fixed; other end translation fixed, rotation free.

From NZS 3404 (SNZ1997) and BHP (1998), the following are obtained:

- Effective length factor, k_e, = 0.85
- Form factor, k_f, = 1.0
- Net area of cross section, A_n = 20100 mm^2 [assuming $A_n = A_g$ (gross area of cross section)]
- Yield stress, flange, (grade 300 steel), σ_y = 280 MPa
- Radius of gyration about minor principal y axis (r_y) = 78.9 mm

Effective length of column, $L_e = k_e L$

$$= 0.85 \times 4.5$$

$$= 3.83 \text{ m}$$

Nominal section capacity, N_s, = $k_f A_n \sigma_y$

$$= 1.0 \times 20100 \times 10^{-6} \times 280 \times 10^3$$

$$= 5628 \text{ kN}$$

Nominal member capacity, N_c, = α_c N_s

Where α_c is the member slenderness reduction factor.

The values of α_c can be obtained from Table 6.3.3(2) of NZS 3404:Part 1:1997 using the value of the modified member slenderness (λ_n) and the appropriate member section constant (α_b) given in Table 6.3.3(1) of the same Standard.
The modified member slenderness (λ_n) is given by the following equation:

$$\lambda_n = \left(\frac{L_e}{r}\right)\sqrt{k_f}\sqrt{\left(\frac{\sigma_y}{250}\right)} \tag{8.5}$$

Therefore, for this particular column, the value of λ_n is given by:

$$\lambda_n = \left(\frac{3.83}{0.0789}\right)\sqrt{1.0}\sqrt{\left(\frac{280}{250}\right)}$$

$$= 51.4$$

From Table 6.3.3(1) of NZS 3404:Part 1:1997, for universal column and beam, hot rolled, flange thickness up to 40 mm and k_f = 1.0, the value of α_b = 0.

From Table 6.3.3(2) of NZS 3404:Part 1:1997, for λ_n = 51.4 and α_b = 0, the value of α_c = 0.854.

The nominal member capacity of the column, N_{cy} $= \alpha_c N_s$

$= 0.854 \times 5628$ kN

$= 4806$ kN

From the above it can be seen that the selected column (310 UC 158) has a nominal member capacity (4806 kN) which is much greater than the required nominal member capacity (1756 kN) under the design gravity loading condition..
The same analysis is done for other beams in the fire compartment and the results tabulated in Table 8.2.

Table 8-2 Performance of Columns under Normal Condition

Column	Nominal member capacity (kN)	Required nominal capacity (kN)	Remark
CC" :310 UC 137 (interior)	4175	1756	ok
DD' :310 UC 137 (edge)	4175	1232	ok
AA' :310 UC 158 (edge)	4806	1224	ok
BB' :310 UC 158 (corner)	4805	670	ok

Evaluation of structural members performance under gravity loading at normal temperature show that these members have been more than adequately designed for such loading. It is now necessary to evaluate similar performance under fire condition.

8.2.4 Structural Response under Fire Condition

A fully developed fire in a building is now regarded as an ultimate limit state event in structural design. Clause 2.4.3.4 (a) of NZS 4203:1992 states that for that period of time during fire emergency conditions when the structure is subject to elevated temperature and designated members are required to remain stable, the affected members shall be designed for the following combination of factored load:

$$L_u = G + Q_u \tag{8.6}$$

This load combination is to be used in structural analysis to ensure that all members and connections shall have the design capacities that satisfy the ultimate limit state equation:

$$S^* \leq \phi R_u \tag{8.7}$$

Where (for structural fire design):

S^* is the design action for the load combination $G + Q_u$ during fire emergency situations.

ϕR_u is the design capacity of the member at elevated temperature

8.2.4.1 Temperature Domain Analysis

The difficulty encountered in calculating the design capacity of the member at elevated temperature (ϕR_u) is that the temperature attained by the member has to be determined first, and the material properties at this temperature should be known. To add to the difficulty, it is known that under fire condition both the material temperature and properties vary with the fire severity. However, it is also known that for a member acted upon by a fixed load value, there exists a critical material temperature beyond which the member is not able to support this fixed value of load. This critical temperature is also called the limiting temperature. Hence, a limit state equation that satisfies the global limit state equation is of the form:

$$T^* \leq T_l \tag{8.8}$$

Where (for structural fire design):

T^* is the maximum temperature calculated to be attained by steel member under the fire condition with passive protection (if any).

T_l is the limiting steel temperature under load condition for fire

This will be the basis of the temperature domain analysis of the structural performance of the steel members under fire condition.

8.2.4.2 Time Domain Analysis

Performance of building element can also be analysed in the time domain under the concept of fire resistance rating. The fire resistance rating for the steel members will not be used here because they are unknown or are not provided by the manufacturers. Instead the fire resistance rating is defined here as the "time to reach limiting temperature." The "time to reach limiting temperature" can be compared with the "time equivalence for fire severity" to obtain the limit state equation:

$$t_e \leq t_l \tag{8.9}$$

where: t_l is the time to reach limiting temperature (fire resistance)

t_e is the time equivalence (fire severity)

To carry out the analyses under both the temperature and the time domains it is necessary to set up the fire model, the heat transfer model and the structural response model. These are described in the following sections.

8.2.5 The Fire Model

8.2.5.1 Parametric Fire Time Temperature Curves

The parameters used determination of the Eurocode parametric fire are shown in Table 8-3.

Table 8-3 Parameters Used for Modeling Parametric Fire

Parameters	Values
Room Depth (m)	8.44
Room Width (m)	3.58
Room Height (m)	2.40
Area of Vertical Openings (m^2)	4.30
Weighted Average Height of Vertical Openings (m)	1.54
Thermal Inertia of Lining Material ($J/m^2Ks^{0.5}$)	1160
Fuel Load Density (MJ/m^2 floor area)	400

Explanatory Notes

- The room height is the height to ceiling and is used for calculating the total internal surface area of the room
- The vertical openings include the windows and doors to corridor and balcony
- The thermal inertia of lining material, b, is assumed to be equal to 1160 $J/m^2Ks^{0.5}$, which is considered reasonable for small enclosures with walls lined primarily by drywall gypsum board, HERA (1995)
- The fuel load density of $400MJ/m^2$ (floor area) is the same as the value given in Acceptable Solution C3/AS1, for Fire Hazard Category 1, which includes hotel, BIA (1995)

The Eurocode (EC1 1994) gives an equation for "parametric" fire, allowing a time-temperature relationship to be produced for any combination of fuel load, ventilation openings and wall lining materials.

The time-temperature curve in the heating phase, from $t^* = 0$ to $t^* = t_d^*$, is given by:

$$\theta_g = 1325(1 - 0.324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.472e^{-19t^*}) \quad (8.10)$$

$$t^*_d = (0.13 \times 10^{-3} q_{t,d}\Gamma)/O \quad (8.11)$$

Where;

θ_g = air temperature in the fire compartment (°C)

t^* = $t.\Gamma$, fictitious time (h)

t = time (h)

Γ = $(O/b)^2/(0.04/1160)^2$

O = $A_v\sqrt{h}/A_t$, the opening factor, such that $0.02 \le O \le 0.20$ ($m^{0.5}$)

A_v = area of vertical opening (m^2)

h = weighted average height of vertical opening (m)

A_t = total internal surface area of compartment, including openings (m^2)

b = $\sqrt{(k\rho c)}$, thermal inertia of lining material, such that $1000 \le b \le 2000$ ($J/m^2Ks^{0.5}$)

k = thermal conductivity of lining material (W/mK)

ρ = density of lining material (kg/m^3)

c = specific heat capacity of lining material (J/kgK)

$q_{f,d}$ = fuel load density based on compartment floor area (MJ/m^2)

$q_{t,d}$ = fuel load density (total internal surface area of compartment) (MJ/m^2)

The cooling phase is from $t^* > t_d^*$ and the time-temperature curve in the cooling phase is given as:

For $t_d^* \leq 0.5$ hours, $\theta_g = \theta_{max} - 625(t^* - t_d^*)$ (8.12)

For $0.5 < t_d^* < 2$ hours, $\theta_g = \theta_{max} - 250(3 - t_d^*)(t^* - t_d^*)$ (8.13)

For $t_d^* \geq 2$ hours, $\theta_g = \theta_{max} - 250(t^* - t_d^*)$ (8.14)

Where θ_{max} = maximum air temperature in the heating phase (°C), for $t^* = t_d^*$

8.2.5.2 Modified Cooling Phase (Buchanan)

Buchanan (1999) stated that there are a lot of confusion regarding the Eurocode expression for the temperature decay or cooling phase because it is given in terms of fictitious time rather than real time. This appears to be a mistake because it gives extremely fast decay rates for large opening factors in well insulated compartments and extremely slow decay rates for small opening factors in poorly insulated compartments. The following decay rate related to real time is recommended:

For $t_d \leq 0.5$ hour, $d\theta_g/dt = 625°C$ per hour (8.15)

For $t_d \geq 2$ hours, $d\theta_g/dt = 250°C$ per hour (8.16)

For $0.5 < t_d < 2$ hours, linear interpolation can be carried out between the above

Where: t_d, the heating duration in real time, $= 0.00013 q_{t,d}/O$ (h).

For the purpose of structural design it is recommended to use a decay rate $d\theta_g/dt$ of 625°C per hour modified for opening factor and thermal insulation, given by

$$d\theta_g/dt = 625\,(O/0.04)(\sqrt{(k\rho c_p)}/1160) \qquad (8.17)$$

These decay rates are based on some burning continuing in the decay phase, which is consistent with the duration in equation (8.11) being less than the theoretical duration.

8.2.5.3 Modified Cooling Rate (HERA)

The Eurocode expression for the cooling phase has also been modified by the New Zealand Heavy Engineering Research Association to more accurately represent experimental results (HERA 1995). The modified Eurocode expression, which gives a more conservative result than that obtained from the original expression, is given below.

For $0.08 \leq O \leq 0.20\ m^{0.5}$

- For $t^* = t_d^*$ to $t^* = t^*_{\theta=600°C}$

for $t_d^* \leq 0.5$ hours, $\theta_{gc1} = \theta_{max} - 625(t^* - t_d^*)$ (i)

for $0.5 < t_d^* < 2$ hours $\theta_{gc1} = \theta_{max} - 250(3 - t_d^*)(t^* - t_d^*)$ (ii)

for $t_d^* \geq 2$ hours, $\theta_{gc1} = \theta_{max} - 250(t^* - t_d^*)$ (iii)

where:

$t^*_{\theta=600°C}$ = value of t* corresponding to $\theta_{gc} = 600°C$ as determined from equations (1) to (3) above

θ_{max} = maximum temperature in the heating phase (°C)

- For $t^* = t^*_{\theta=600°C}$ to $t^* = t^*_{\theta=400°C}$

$$\theta_{gc2} = 600 - 7080\, O(t^* - t^*_{\theta=600°C})/\Gamma \quad \text{(iv)}$$

where:

$t^*_{\theta=600°C}$ = as defined above

$t^*_{\theta=400°C}$ = value of t* corresponding to $\theta_{gc} = 400°C$ as determined from equation (4) above

- For $t^* = t^*_{\theta=400°C}$ to $t^* = t^*_{\theta=250°C}$

$$\theta_{gc3} = 400 - 1680\, O(t^* - t^*_{\theta=400°C})/\Gamma \quad \text{(v)}$$

where:

$t^*_{\theta=400°C}$ = as defined above

$t^*_{\theta=250°C}$ = value of t* corresponding to $\theta_{gc} = 250°C$ as determined from equation (v) above

- For $t^* = t^*_{\theta=250°C}$ to ambient temperature

Decrease temperature by 10°C per minute down to ambient temperature

For $0.04 \leq O \leq 0.08m^{0.5}$

- For $t^* = t_d^*$ to $t^* = t^*_{\theta=600°C}$

  Use equations (i) to (iii) above

- For $t^* = t^*_{\theta=600°C}$ to ambient temperature

Use equation (iv) above

For $O < 0.04m^{0.5}$

Use equation (i) to (iii) for the entire cooling phase.

For comparison purpose, all the three expressions for the cooling phase are used in the spreadsheet for parametric fire time temperature curves and the resulting curves are shown in Fig. 8-2

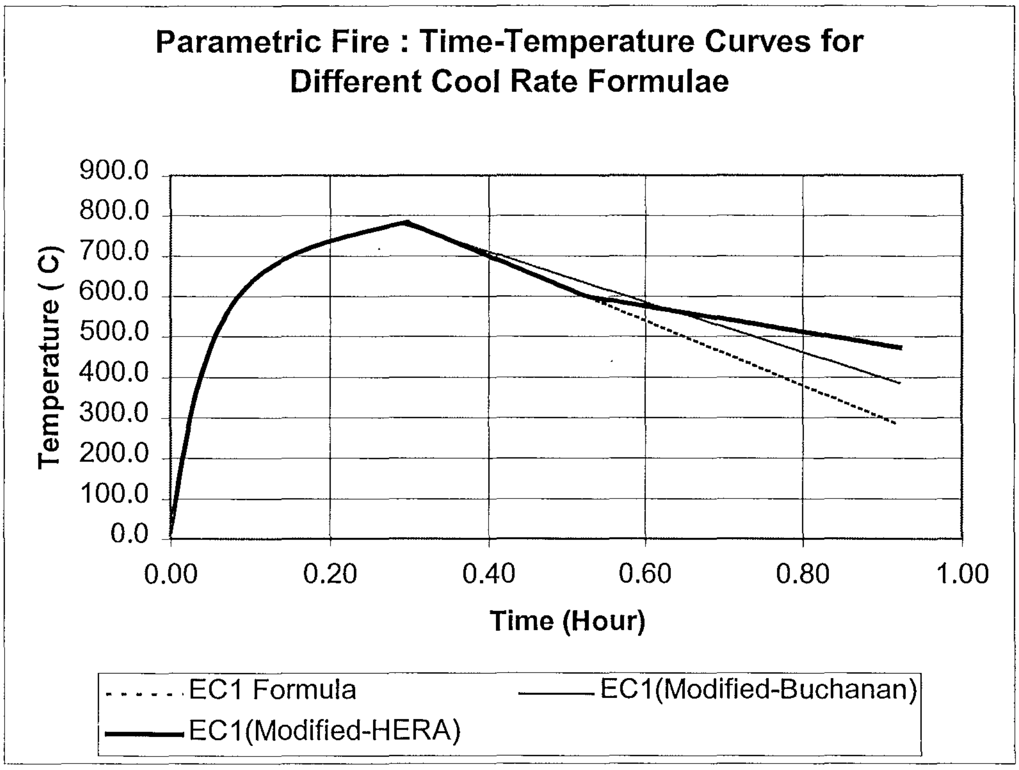


Figure 8-2 Time-Temperature Curves for Different Cooling Rate Formulae

The EC1 (Modified-HERA) cooling curve was used in the original design by the consultant firm. For consistency it will also be used in this project.

8.2.6 The Heat Transfer Model

8.2.6.1 Determination of Steel Temperature

The temperature domain verification is used when the limiting steel temperature is compared with the maximum temperature reached in the design fire exposure. The limiting steel temperature is the temperature at which the load-bearing capacity of the member would just equal the design loads. In other words, it is the temperature above which the member would be expected to fail. Heat transfer calculation to determine steel temperature is simplified by using a lumped mass thermal calculation where the steel cross section is assumed to be at uniform temperature. The European Convention for Constructional Steelwork (ECCS 1985) recommends the use of the following equations for time-temperature curves for protected and unprotected steel.

8.2.6.2 Unprotected Steel

The iterative calculation technique for unprotected steelwork uses the following equation:

$$\Delta T_s = \frac{h_t}{\rho_s c_s} \frac{H_p}{A} (T_f - T_s) \Delta t \qquad (8.18)$$

where ΔT_s is the change in steel temperature over the time interval Δt (°C)

h_t is the total heat transfer coefficient (W/m^{2}K)

ρ_s is the density of steel (kg/m^3)

c_s is the specific heat of steel (J/kgK)

H_p/A is the exposed surface area to mass ratio for the steel section (m^{-1})

T_f is the mean air temperature during time interval Δt (°C)

T_s is the mean steel temperature during time interval Δt (°C)

Δt is the time interval (s)

The total heat transfer coefficient h_t is the sum of the radiative and convective heat transfer coefficients:

$$h_t = h_c + h_r \qquad (8.19)$$

Where h_c is the convective heat transfer coefficient (W/m^2K)

h_t is the radiative heat transfer coefficient (W/m^2K)

The Eurocode (EC1 1994) recommends that the value of the convective heat transfer coefficient be taken as $25 W/m^2K$ for the standard fire and $50 W/m^2K$ for the hydrocarbon fire.

The radiative heat transfer coefficient is defined as:

$$h_r = \frac{\sigma\varepsilon(T_f^4 - T_s^4)}{(T_f - T_s)}. \qquad (8.20)$$

Where: σ is the Stefan Boltzman constant ($56.7 \times 10^{-12}\ kW/m^2K^4$)

ε is the resultant emissivity

T_f is the temperature in the fire compartment (K)

T_s is the temperature of the steel (K)

Eurocode1 (EC1 1994) recommends a value of resultant emissivity of 0.56 which is a product of the emissivity related to the fire compartment, usually taken as 0.8, and the emissivity related to the surface material, usually taken as 0.7.

A computer spreadsheet program can be used in a step-by-step calculation technique assuming a lumped mass of steel at uniform temperature. A spreadsheet for calculating steel temperature using this approach is shown in Table 8.4 (from Milke and Hill, 1996, based on Gamble, 1989). EC2 (1995) suggests a time step of no more than 30 seconds, and a minimum value of the section factor H_p/A of $10m^{-1}$.

Table 8-4 Spreadsheet Calculation for Heat Transfer in Unprotected Steel Members

Time	Steel temperature T_s	Fire temperature T_f	Temperature Difference $T_f - T_s$	Heat transfer coefficient h_t	Change in steel temperature ΔT_s
$t_1 = \Delta t$	Initial steel temperature T_∞	Fire temperature at $t = \Delta t/2$	$T_f - T_s$	Calculate h_c from T_f and T_s $h_t = h_r + h_c$	Calculate from equation for ΔT_s
$t_2 = t_1 + \Delta t$	$T_s + \Delta T_s$ from previous row	Fire temperature at $t = t_1 + \Delta t/2$	$T_f - T_s$	Calculate h_c from T_f and T_s $h_t = h_r + h_c$	Calculate from equation for ΔT_s
etc					

The detailed spreadsheet set up is described in Appendix I

8.2.6.3 Protected Steel

The iterative technique for protected steelwork is similar to that for unprotected steel. The equation is slightly different and does not require heat transfer

$$\Delta T_s = \frac{H_p}{A}\frac{k_i}{d_i \rho_s c_s}\left\{\frac{\rho_s c_s}{\left(\rho_s c_s + \frac{H_p}{A} 2 d_i \rho_i c_i\right)}\right\}\left(T_f - T_s\right)\Delta t \qquad (8.21)$$

coefficients because it assumes that the external surface of the insulation is at the same temperature as the fire gases. It also assumes that the internal surface of th (8.19) insulation is at the same temperature as the steel. The equation is:

Where k_i is the thermal conductivity of the insulating material

d_i is the thickness of the insulating material

ρ_i is the density of the insulating material

c_i is the specific heat of the insulating material

If the insulation is of low mass and specific heat such that the heat capacity of the insulation will not significantly slow the temperature increase of the steel, then the above equation can be simplified by omitting the term in the big {} brackets (ECCS 1985). That is, for lightly insulated members, the equation for the steel temperature rise is given by:

The spreadsheet calculation is similar to that shown in Table 8-2 except that no heat transfer coefficient is required. EC2 (1995) suggests a time step of 30 seconds, but Gamble (1989) shows that much longer steps can be used.

8.2.7 Scenarios in Present Analysis

In this project the original design by the consultant firm will be modelled as one scenario. In addition, two other scenarios are also modelled. The three scenarios are described below.

8.2.7.1 Scenario 1: Original Design - Members Shielded by Ceiling or Wall Linings

In the original design no passive protection is directly applied to the beams and columns. The suspended "Gib-Grid' ceiling system (which shields the beams) and the Gib-board wall claddings (which shield the columns) are the only passive protection. Both the ceiling and the wall claddings are assumed to fail 20 minutes after flashover. It is also assumed that within the 20 minutes before failure of the ceiling or wall claddings, the steel members are considered to be lightly protected and equation (8.22) is applicable. After the failure of the ceiling and wall claddings, the steel members are considered to be unprotected and equation (8.18) applies.

The "Gib-Grid" ceiling system uses standard 12.5mm Gib-board with taped and stopped joints. The Gib-board wall claddings are all 9.5mm thick. The thermal conductivity of the Gib-board is assumed to be 0.20 W/mK

8.2.7.2 Scenario 2: Totally Unprotected Steel Members

A model is also set up for the case where all unprotected steel members in the fire compartment, without the benefits of shielding from the ceiling or wall claddings, are fully exposed to the fire.

8.2.7.3 Scenario 3: Steel Members Protected by Sprayed on Mineral Fibre

In the same manner, a model is also set up for the case of steel members protected by sprayed on mineral fibre, again without the benefits of shielding from the ceiling and wall claddings. Equation (8.21) is used for the heat transfer calculation. The properties of the mineral fibre are assumed to be (Buchanan 1999):

Density: 300 kg/m^3

Thermal Conductivity: 0.10 W/mK

Specific heat: 1100 J/kg K

Water content: negligible

8.2.8 Properties of Structural Steel at Elevated Temperatures

In order to make calculations of temperatures in fire-exposed steel structures, it is necessary to know the properties of the structural steel at elevated temperatures.

8.2.8.1 Thermal Properties

The density of steel is 7850 kg/m^3, remaining essentially constant with temperature. The specific heat of steel varies with temperature but for simple calculations the value can be taken as 600J/kg K.

8.2.8.2 Mechanical Properties

The important mechanical properties of structural steel required for structural fire design are the yield strength and the modulus of elasticity. The variation of the yield strength and modulus of elasticity with temperature are shown in figure 8-3.

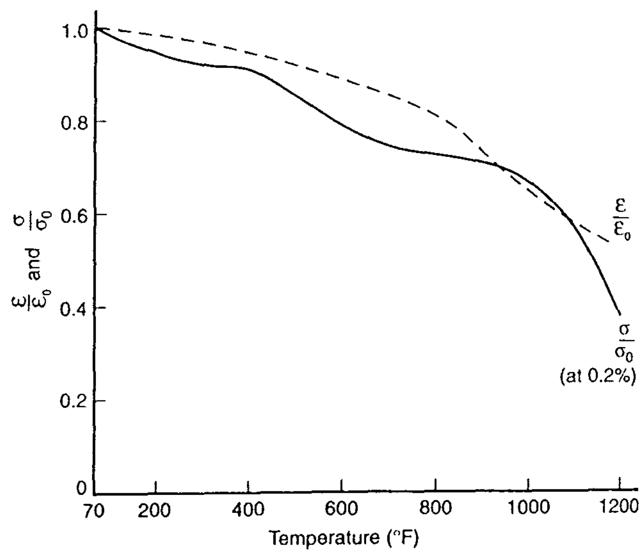


Figure 8-3 Variation of Modulus of Elasticity and Yield Strength with Temperature A36 Steel (DeFalco 1974)

Many national codes use approximations to the published data on actual variation of properties. Typical relationships are shown in Figure 8-4, where the line for structural steels is from AS 4100 and NZS 3404, and the lines for reinforcing steel and prestressing steel are from BS 8110, AS3600 and NZS 3101.

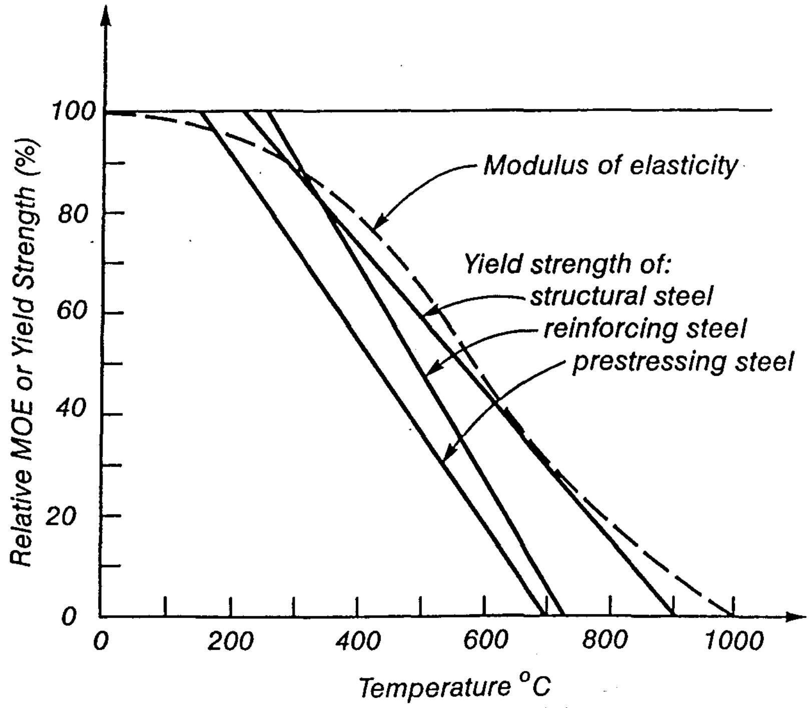


Figure 8-4 Reduction in Yield Strength and Modulus of Elasticity with Temperature - Design Curves (Buchanan 1999)

The equations of the lines are:

$K_{y,T}$ = (905 - T)/690 for structural steel (8.23a)

$K_{y,T}$ = (720 - T)/470 for reinforcing steel (8.23b)

$K_{y,T}$ = (700 - T)/550 for prestressing steel (8.23c)

Where $K_{y,T} = f_{y,T}/ f_y$

$f_{y,T}$ is the yield strength at temperature T (°C)

f_y, is the yield strength at 20°C

8.2.9 Determination of Limiting Steel Temperature

The limiting steel temperature can be defined as the highest temperature attained by the steel member, when exposed to a fire, beyond which the member is not able to support the existing loadings. The limiting steel temperature depends not only on the steel properties but also on the load and support conditions, dimensions, and geometry of the structural member.

In clause 11.5 of NZS 3404 (SNZ1997), the equation for determining the limiting steel temperature, which originates from equation (8.23a), states that the limiting steel temperature T_l (°C) is given by:

$$T_l = 905 - 690\, r_f \tag{8.24}$$

Where r_f is the ratio of the design action on the member under the design load for fire (G + Q_u) to the design capacity of the member (ϕR_u) at room temperature. Equation 8.24 is derived directly from equation 8.23a.

8.2.10 Time to Attain Limiting Temperature

The time for steel members to reach limiting temperature are obtained by two methods :

(a) Empirical Correlations

Simple empirical expressions for predicting the time t_l in minute for an unprotected steel member to reach a limiting temperature T_l (°C) when exposed to the standard fire are given in NZS 3404 (SNZ 1994) as follows:

For 3 sided exposure

$$t_l = -5.2 + 0.0221T_l + 0.0\text{-}3.40\ T_l\ V/F \tag{8.25}$$

For four sided exposure

$$t_l = -4.7 + 0.0263T_l + 1.67T_l\ V/F \tag{8.26}$$

Where: F is the surface area of the steel section, per unit length (m^2)

V is the volume of steel per unit length (m^3)

Both of these equations are valid for F/V in the range 15 to 275 m^{-1} (V/F in the range 3.6 to 67 mm) and T_l in the range 500°C to 800°C. For temperature below

500° C linear interpolation can be used based on the time at 500°C and the initial temperature 20°C.

(b) Spreadsheet Calculation

Equation (8.24) for determining limiting steel temperature has been input into the spreadsheet. Also included in the same spreadsheet are the time-temperature curves for steel members (protected or otherwise) exposed to the standard fire. The time taken for these steel members to attain T_l can therefore be obtained from the spreadsheet.

8.2.11 Determination of Time Equivalence for Fire Severity

In this project, the time equivalence is calculated by two methods:

(a) Spreadsheet Method

The maximum temperature of steel members protected by sprayed on mineral fibre and exposed to parametric fire, T_{max}, is first obtained from the spreadsheet. The same protected member is then exposed to the standard fire and the time taken, t_e , for the steel to reach the temperature T_{max} is obtained from the spreadsheet. The time t_e is the time equivalence.

(a) **Eurocode Formula**

The time equivalence is also calculated using the Eurocode formula (equation 4.6) which has been described in chapter 4.

8.3 THE COMPLETE WORKING MODEL

The fire model, heat transfer model and the structural response models have been combined into a single complete working model that can be used for structural fire design or to assess the responses of existing structures. The spreadsheet set up for the complete working model and results of calculations are appended in Appendix I.

8.4 RESULTS

Analyses are carried out in both the temperature and time domains. In each domain, the three scenarios for the steel members are analysed :

(i) unprotected,

(ii) unprotected but shielded by suspended ceiling or wall cladding

(iii) protected by sprayed on mineral fiber (ceiling and wall not accounted for)

8.4.1 Temperature Domain Analysis

(i) Unprotected Steel Members

The limiting and maximum temperature for unprotected steel members is shown in Table 8.5.

Table 8-5 Limiting and Maximum Steel Temperature of Unprotected Steel Members

Steel Members	Limiting Steel Temperature (°C)	Maximum Steel Temperature (°C)	Remark
AB: 460 UB 74.6	800	744	ok
AC: 460 UB 74.6	810	754	ok
BD: 460 UB 74.6	839	754	ok
EF: 310 UB 40.4	666	765	failed

It can be seen from Table 8-1 that for the case of unprotected steel members fully exposed to the fire in the fire compartment only one member fails under the temperature criterion. The member is the intermediate beam EF that supports the concrete slab.

Unprotected but Shielded Steel Members

The results for the case of unprotected members shielded for 20 minutes by either the suspended ceiling or wall claddings ("shielded steel members") are shown in Table 8.6.

Table 8-6 Limiting and Maximum Steel Temperature of "Shielded" Steel Members

Steel Members	Limiting Steel Temperature (°C)	Maximum Steel Temperature (°C)	Remark
AB: 460 UB 74.6	800	628	ok
AC: 460 UB 74.6	810	643	ok
BD: 460 UB 74.6	839	643	ok
EF: 310 UB 40.4	666	664	ok
AA': 310 UC 158	806	550	ok
BB':310 UC 158	848	550	ok
CC': 310 UC 137	711	576	ok
DD': 310 UC 137	793	576	ok

As can be seen from Table 8.6, the maximum steel temperatures of "shielded" members exposed to the parametric fires are all less than the limiting steel temperatures. From these deterministic calculations, the steel members are deemed to possess the required fire resistance under the present conditions.

Protected Members

The case of the steel members protected by sprayed on mineral fibers and exposed to the same fire is shown in Table 8-7.

Table 8-7 Limiting and Maximum Steel Temperature of Protected Members

Steel Members	Limiting Steel Temperature (°C)	Maximum Steel Temperature (°C)	Remark
AB: 460 UB 74.6	800	435	ok
AC: 460 UB 74.6	810	452	ok
BD: 460 UB 74.6	839	452	ok
EF: 310 UB 40.4	666	476	ok
AA': 310 UC 158	806	335	ok
BB':310 UC 158	848	335	ok
CC': 310 UC 137	711	371	ok
DD': 310 UC 137	793	371	ok

Results in Table 8-7 show that the maximum temperature attained by all the protected steel members are well below the limiting steel temperature. The protective coating has ensured safe performance.

8.4.2 Time Domain Analysis

The three scenarios will be looked into viz (i) unprotected, (ii) shielded and (iii) protected by sprayed on mineral fiber. The results, for different methods of calculating "time to limiting temperature" and the "time equivalence", are shown and discussed in the next few pages.

(i) Unprotected Steel Members' Performance

The results for unprotected steel members are shown in Table 8-8

The formula used for obtaining time to limiting steel ($t_{l/s}$) temperature is either equation (8.25) or (8.26).

Table 8-8 Unprotected Steel Members' Performance under Time Domain

	Time to reach limiting temperature (min)		Time Equivalence (min)		
Steel Member	Spreadsheet $t_{l/s}$	Formula $t_{l/f}$	Spreadsheet $t_{e/s}$	EC Formula $t_{e/f}$	Remark
Beam AB	27	30	36	29	Failed
Beam AC	27	29	35	29	Failed
Beam BD	32	30	35	29	Failed except when $t_{e/f}$ is used
Beam EF	15	20	33	29	Failed
Column AA'	33	32	40	29	Failed except when $t_{e/f}$ is used
Column BB'	38	34	40	29	Failed except when $t_{e/f}$ is used
Column CC'	23	25	38	29	Failed
Column DD'	29	27	38	29	Failed

Note: In all cases, spreadsheet calculation for the time equivalence is based on steel members protected by sprayed on mineral fibre and exposed to parametric and standard fires

The following can be deduced from Table 8-8

- All unprotected steel members will fail, except in some cases when the Eurocode formula for time equivalence is used
- The formulae for calculating "time to limiting temperature" (equation 8.25 and 8.26) agree well with the spreadsheet calculations

Unprotected but Shielded Members

The results for the case of steel members shielded for 20 minutes by suspended ceiling or wall claddings are shown in Table 8-9.

Table 8-9 "Shielded" Steel Members' Performance under Time Domain

Steel Member	Time to reach limiting temperature (min)	Time Equivalence (min)		Remark
	Spreadsheet $t_{l/s}$	Spreadsheet $t_{e/s}$	EC Formula $t_{e/f}$	
Beam AB	30	36	29	Failed except when $t_{e/f}$ used
Beam AC	30	35	29	Failed except when $t_{e/f}$ used
Beam BD	33	35	29	Failed except when $t_{e/f}$ used
Beam EF	23	33	29	Failed
Column AA'	38	40	29	Failed except when $t_{e/f}$ is used
Column BB'	41	40	29	ok
Column CC'	30	38	29	Failed except when $t_{e/f}$ used
Column DD'	34	38	29	Failed except when $t_{e/f}$ used

The following can be deduced from Table 8-9:

- Using suspended ceiling or wall claddings (with "fire resistance rating" of 20 minutes) as shields for unprotected steel members have not enhanced their performance appreciably.
- Eurocode formula for time equivalence is less conservative compared with the spreadsheet calculations for obtaining time equivalent.

Protected Steel Members

The results for the case of steel members protected by sprayed on mineral fibre is shown in Table 8-10

Table 8-10 Protected Steel Members' Performance under Time Domain

Steel Member	Time to reach limiting temperature (min)		Time Equivalence (min)		Remark
	Spreadsheet $t_{l/s}$	Formula $t_{l/f}$	Spreadsheet $t_{e/s}$	EC Formula $t_{e/f}$	
Beam AB	87	68	36	29	ok
Beam AC	83	63	35	29	ok
Beam BD	90	66	35	29	ok
Beam EF	53	43	33	29	ok
Column AA'	125	109	40	29	ok
Column BB'	138	116	40	29	ok
Column CC'	89	79	38	29	ok
Column DD'	107	91	38	29	ok

Beams AB, AC & BD : 460 UB 74.6; Beam EF: 310 UB 40.4;
Column AA', BB': 310 UC 158; Columns CC' & DD' : 310 UC 137

Data from Table 8-10 clearly shows that, under time domain analysis, all the steel members are only safe if passive protection like a sprayed on coating has been applied on the members.

8.5 DISCUSSION

The results from analyses in the temperature and time domain show that the latter domain produce much more conservative results. The time safety margins are relatively lower than the temperature safety margins. The time domain analyses show that passive protection like sprayed on mineral fibre is required in order to produce safe performance, whereas even unprotected members can be considered safe under fire situation when analysed under the temperature domain. These disparities can be attributed to the following reasons:

- Analysis in the temperature domain is carried out based on exposure only to the parametric fire. Because it is not necessary to compare thermal response to the standard fire, this method can be considered more accurate as less error and uncertainty is introduced.
- For analysis in the time domain the "time to limiting temperature" and the "time equivalence" for fire severity are calculated based on exposure to the standard fire. The parametric fire is only used to determine the maximum steel temperature, the value of which is used in the exposure to the standard fire to obtain the time equivalence. Furthermore, the determination of the time equivalence is only valid for protected steel members. Hence, using the time equivalence (obtained from exposure of protected steel member) for deduction from "time to reach limiting temperature" for unprotected and "shielded" members to obtain the "time safety margin" may incur errors.
- The same problem arises in the use of the Eurocode formula for time equivalence, the derivation of which is based on protected steel exposed to the standard fire.
- For the case of protected steel the problem of comparing results from different "geometries" do not arise. Hence the time safety margin for protected steels agree very well with the temperature safety margin.
- In the temperature domain analysis there are many cases of the maximum steel temperatures going beyond 550°C and are considered safe because the limiting temperatures have not been reached. 550°C is considered by many national codes to be the "critical" steel temperature (Lie 1992) because at around this temperature, failure of structural steel is impending (Fitzgerald 1997) if the member is subjected to its maximum design load. The critical temperature is higher when load levels are lower, as in this example.

- For this reason, it is widely claimed that structural steel works can be designed to withstand temperature up to 750°C without sign of failure (Robinson 1995 and Purkiss 1996). The British Standard allows limiting temperature up to 780°C for structural steel members in flexure (BS5950 1990).
- Temperature domain analysis is a more accurate method for assessing or validating design but the use of numerical methods or computer software to more accurately determine the steel temperature is needed so that a useful and accurate methodology for design can be derived.

8.6 SUMMARY

This chapter has shown how simple analytical tools can be used to check the structural adequacy of steel members exposed to fire. The calculations are limited by the accuracy of methods of compiling the temperature of the steel members. The calculations in the time domain show the principles of the method, but these results are not accurate because of poor temperature calculation and the inherent inaccuracy of the time equivalent formula.

9 CASE STUDY II : RELIABILTY OF STRUCTURAL FIRE DESIGN OF A STEEL FRAMED BUILDING

9.1 BASIS OF RELIABILITY ANALYSIS

In this case study, reliability analysis is carried out on a structural fire engineering design of a steel framed hotel building described in chapter 8. As stated in that chapter, the assessment of the structural performance of steel members under fire condition was carried under both the temperature and time domains. The bases of assessment are recapitulated below.

(a) Temperature Domain Analysis

In the temperature domain analysis, the basis of reliable or safe performance is that the maximum temperature attained by the structural steel member should not exceed the limiting temperature of the member. The limit state equation is thus given by:

$$T_m = T_l - T_{max} \tag{9.1}$$

Where T_m is the temperature safety margin (°C)

T_l is the limiting steel temperature for the steel member (°C)

T_{max} is the maximum temperature attained by the steel element (°C)

Failure occurs when the temperature safety margin is less than or equal to zero, that is:

$$T_M = T_l - T_{max} \leq 0 \tag{9.2}$$

Reliability analysis is carried out on the basis of determining the probability of failure, P_f, of the structural member, given the outbreak of a range of fully developed fires. The probability of failure is the probability of the temperature safety margin being less than or equal to zero, that is:

$$P_f = P(T_m \leq 0) = P(T_l - T_{max} \leq 0) \tag{9.3}$$

Methods of calculating T_{max} and T_l have been described in section 8.3.6 and 8.3.7 respectively.

(b) Time Domain Analysis

In the time domain, the consideration is that the fire resistance rating of the structural element can more than withstand the maximum fire severity (expressed as time equivalence) expected in the fire compartment. The limit state equation is therefore given by:

$$M = R - S \tag{9.4}$$

Where M is the safety margin (minute)

R is the fire resistance rating (min)

S is fire severity (min)

In this project, the fire resistance R is taken as the time for steel member to attain limiting temperature, t_l. And the fire severity S is represented by the time equivalence, t_e. Equation (9.4) can thus be written as:

$$t_m = t_l - t_e \tag{9.5}$$

where t_m is the time safety margin

The failure criterion is given by:

$$t_m = t_l - t_e \leq 0 \tag{9.6}$$

The probability of failure of member, P_f, is thus given by:

$$P_f = P(t_m \leq 0) = P(t_l - t_e \leq 0) \tag{9.7}$$

Methods of calculating t_e and t_l have been described in section 8.3.8 and 8.3.9 respectively.

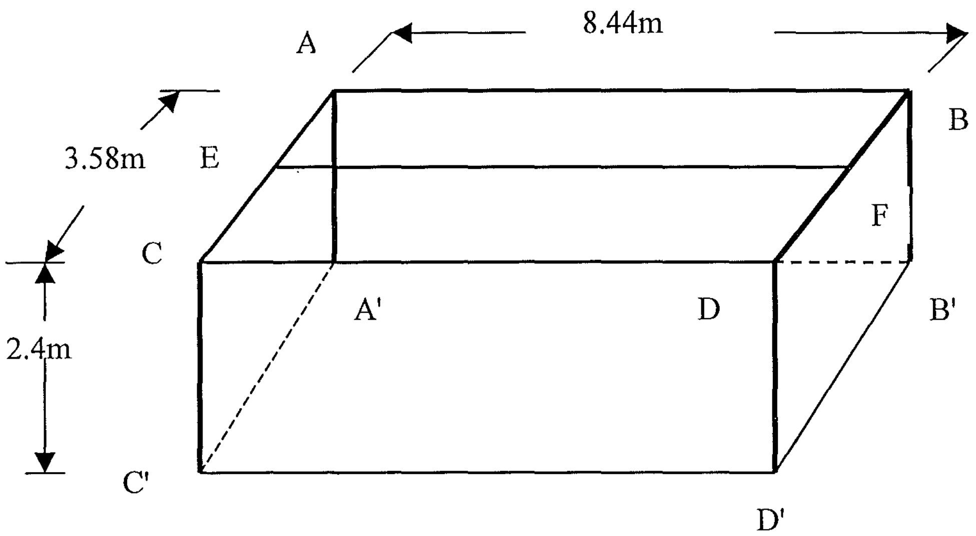
The basis of reliability analysis is the setting up of a model to evaluate equation (9.3) and (9.7) for a range of scenarios. The model should therefore be capable of generating a wide range of scenarios for the fire and the corresponding thermal and structural responses.

As described in chapter 8, a number of parameters have gone into the calculation of T_l,, T_{max}, t_l and t_e. In assessing the reliability of the structural fire design carried out in chapter 8, all these parameters have to be treated as random variables, as they rightly are. Reliability analysis is only carried out on all the structural steel members found in a representative hotel room.

9.2 RELIABILTY ANALYSIS OF DESIGN

9.2.1 Method of Analysis

The tool used here for reliability analysis is Monte Carlo Simulation which has been described in section 6.4.3. As described in chapter 8 the necessary parameters and variables had been entered into computer spreadsheets for determining the air-time temperature curves due to parametric fires in different firecells. The thermal and structural responses of the steel elements were also determined by calculations done on the same spreadsheets. A sketch of the representative fire compartment showing the room dimension and steel member sizes are shown in Fig.9.1.



Beam Size		Column Size	
AB	460 UB 74.6	AA'	310 UC 158
EF/CD	310 UB 40.4	BB'	310 UC 158
AC	460 UB 74.6	CC'	310 UC 137
BD	460 UB 74.6	DD'	310 UC 137

Figure 9-1 Fire Compartment Dimension and Steel Member Sizes

In chapter 8 the spreadsheet calculations have been done deterministically because only single value of each of the parameters has been used. In the Monte Carlo simulation using @RISK software, the same spreadsheet is used for the creation of the model for simulation. The main difference between the @RISK spreadsheet and the deterministic Excel spreadsheet is that the basic parameters in @RISK spreadsheet are treated as random variables and keyed into the cells of the @RISK spreadsheet as probability distribution functions. In treating the parameters as random variables, as they rightly are, it is most important that the probability distribution function chosen should, as accurately as possible, reflect the actual behaviour of these random variables. The setting up of a model for Monte Carlo simulation of the structural response of a steel beam EF (310 UB 40.4) in the representative fire compartment is described below.

9.2.2 Characterisation of Random Variables

The basic variables in the spreadsheet are characterised as random variables as follows.

(a) Room Depth

Variation is mainly due to construction and measurement errors. These errors are estimated to be in the order of 1%. The room depth is most realistically treated as a random variable which exhibits a triangular distribution. Triangular distribution is characterised by three parameters viz the most likely value, a minimum and a maximum value. Both the minimum and maximum value have zero probability of occurring. The nominal depth of 8.440m can be regarded as the most likely value. The addition and subtraction of 1% of the most likely value give the maximum value of 8.524m and a minimum value of 8.356m respectively.

(b) Room Width

Similarly, the room width is treated as a random variable with triangular distribution. The most likely value is nominal width of 3.580m. The minimum value is 3.544m and the maximum value is 3.616m

(c) Room Height

In the same manner, the room height exhibits a triangular distribution with a most likely value of 2.400m, a minimum value of 2.376m and a maximum of 2.424m

(d) Window Area

The average window area has been worked out from all rooms of a particular design. There is more variation in this random variable and it is assumed to exhibit a normal distribution. The mean value of the window area is taken to be $4.3m^2$ and, assuming a coefficient of variation of 0.10, the standard deviation worked out to be $0.430m^2$.

(e) Window Height

The weighted average height is assumed to be normally distributed and the nominal window height taken as the mean value of a normal distribution with a coefficient of variation of 0.05. The mean height is 1.540m with a standard deviation of 0.077m

(f) Thermal Inertia ($\sqrt{(k\rho c_p)}$)

A value of 1160 $J/m^2s^{0.5}K$ is recommended by HERA for small enclosures with over 50% of the wall area in drywall gypsum board construction (HERA 1995). This value is taken as the mean value of the thermal inertia, which is assumed to be normally distributed with a coefficient of variation assumed to be 0.10. The standard deviation is therefore $116J/m^2s^{0.5}K$.

(g) Fuel Load Density

A value of $400MJ/m^2$ of floor area was used in the deterministic spreadsheet calculation. This is the design value recommended in the Annex to Fire Safety Documents of the Approved Documents (BIA 1995b) for fire hazard category 1, which includes hotel. However, this design value is the 80th percentile fire load of the range of fire load density stated for the relevant fire hazard category. The variation in the fuel load density can be assumed to be normally distributed with a coefficient of variation of 25% (ref European 1986). As such the mean value of the fuel load density is worked to be 330MJ/$m^{2.}$ The standard deviation is therefore 83MJ/m^2.

(h) Steel Section Factor H_p/A

The section factors are normally supplied by steel manufacturers and are not expected to show much variation. However, values supplied by different manufacturers for the same nominal section do differ slightly. The section factor is assumed to exhibit triangular distribution with a minimum value of 180, most likely value of 210 and maximum value of 230 m^{-1}.

(i) Steel Density

Steel density is not expected to show much variation at normal temperature but will vary with temperature. The steel density is treated as a random variable which has a normal distribution between a minimum of 7810kg/m^3 and maximum of 7890kg/m^3. The mean of the truncated normal distribution is 7850kg/m^3 with a standard deviation of 40kg/$m^{3.}$

(j) **Specific Heat of Steel**

In general, the specific heat of steel shows a gradual increase with temperature. For simple calculation, the specific heat is usually taken as 600J/kgK. (Buchanan 1997). To account for the variation, the value of the specific heat is assumed to be normally distributed with a mean of 600J/kgK, and a coefficient of variation of 0.05. The minimum value of the specific heat is taken as 540KJ/kgK and the maximum value 660KJ/kgK.

(k) Concrete Slab Thickness

The concrete slab thickness is expected to be uniform with any variation due to the casting process and shrinkage. The nominal thickness of the concrete slab is 130mm, which can be regarded as the mean value of the assumed normal distribution of the slab thickness. The coefficient of variation is small and assumed to be 0.02. The truncated normal distribution is assumed to have a minimum value of 124mm and a maximum value of 136mm. The standard deviation is 3mm.

(l) Density of Concrete

Density of concrete varies widely, depending on the aggregate mix, and can range from1900kg/m^3 to 2500kg/m^3. A truncated normal distribution between these lower and upper values with a mean of 2200kg/m^3 and standard deviation of 110kg/m^3 is assumed.

(m)Weight of Wall and Partition (Component of Dead Load)

A truncated normal distribution with a mean of 0.50KPa, coefficient of variation of 0.10, minimum value of 0.40KPa and a maximum of 0.70KPa would accurately describe the distribution.

(n) Self Weight of Steel Beam (Component of Dead Load)

The self weight of steel section is usually supplied by the manufacturer and the assumed distribution is a truncated normal distribution with a mean of 0.404KN/m, coefficient of variation of 0.05, minimum value of 0.364KN/m and a maximum value of 0.444KN/m.

(o) Live Load

Loads are due to people, their possessions, storage materials etc. Live load may be divided into two categories:

1. Sustained live loads: long-term loads associated with normal use
2. Extraordinary loads: short-term transient loads caused by abnormal events (e.g a big party)

The total live load is the sum of these two live load components. Each can be represented by a discrete stochaistic process, as illustrated in Fig 9.1.

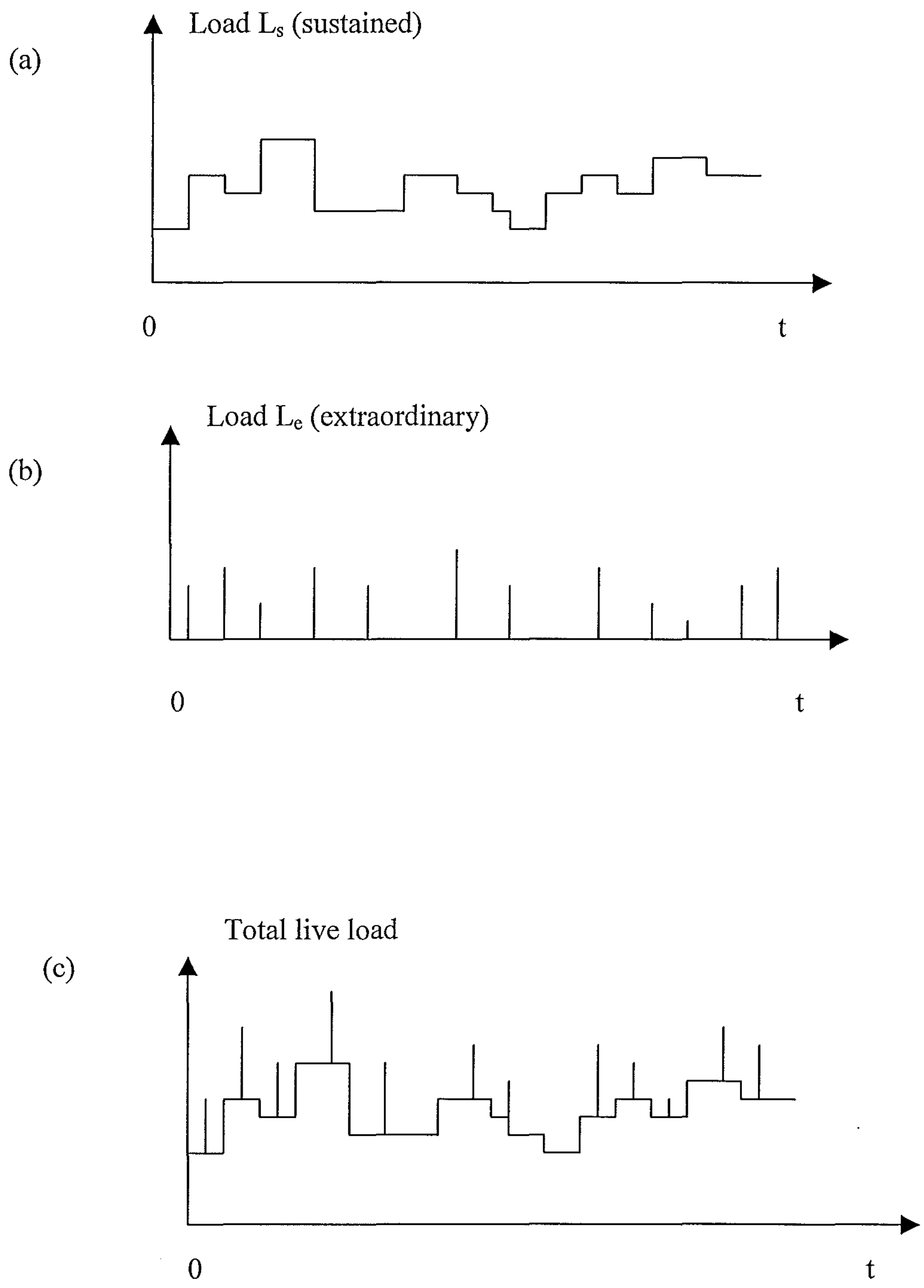


Figure 9-2 Time Histories of Typical Live Loads

Source: Melchers (1987)

The value of the sustained live load and its probability distribution function are arrived at as follows.

From NZS 4203: 1992 Table 3.4.1- Basic Live Loads for Floors and Stairs, the basic live load, Q_b, for hotel occupancy is taken as 2.0KPa.

Reduced live load, Q, is given by $Q = \psi_a Q_b$ where ψ_a is the area reduction factor.

For a tributary area, $A_t = 15.1\text{m}^2$, the area reduction factor is given by

$1 \geq \psi_a = 0.4 + 2.7/\sqrt{A_t}$, that is, $\psi_a = 0.4 + 2.7/\sqrt{15.1} = 1.09$

Therefore $\psi_a = 1$.

That is $Q = 1 \times 2.0 = 2.0$ kPa

The factored live load for the ultimate limit state, Q_u, is given by $Q_u = \psi_u Q$ where ψ_u is the live load combination factor and takes a value of 0.4.

Therefore $Q_u = 0.4 \times 2.0 = 0.8$ kPa

In the deterministic calculation of strength and stability under fire condition, the load combination or the fire design load, L_f, is given by $L_f = G + Q_u$ where G is the dead load.

In a probabilistic calculation, the factored live load Q_u has to be treated as a random variable with a probability distribution function. As to the type of distribution the answer is given in commentary 2.4.3.2 in NZS 4203:1992 Vol. 2 which states in part:

> The specified values of live load are meant to be the peak values of live load for a 50 year life time with a 5% chance of exceedance.

This means that the live load exhibits an extreme value type I distribution of the form: $F_Y(y) = \exp[-e^{-\alpha(y-u)}]$ ref: Ang and Tang (1985)

Where Y is the random variable "live load under normal condition" and y is the value of the random variable.

The same distribution is assumed for the random variable "live load under fire condition" denoted by Y_f. The distribution for Y_f is given by:

$$F_{Yf} = \exp[-e^{-\alpha(yf - u)}]$$

For $y_f = 0.8$kPa, $F_{Yf}(0.8) = \exp[-e^{-\alpha(0.8 - u)}]$

5% chance of exceedance in 50 years means that the chance of exceedance in a year = 5/50 = 0.1%

0.1% chance of exceedance $\Rightarrow \exp[-e^{-\alpha(0.8 - u)}] = 1 - 0.001$

$$\exp[-e^{-\alpha(0.8 - u)}] = 0.999 \tag{1}$$

Also, for Type I distribution, the mean m_Y and the standard deviation σ_Y for a 1 year distribution is given by:

$$m_Y = u + \varphi/\alpha \qquad (2)$$

where φ = Euler constant $\cong 0.577$

$$\text{and} \quad \sigma_Y = \pi/(\alpha\sqrt{6}) \cong 1.283/\alpha \qquad (3)$$

Ellingwood et al (1980) reported a coefficient of variation of 0.15 for Type I distribution live load.

Therefore $\sigma_Y/m_Y = 0.15$

$$1.283/(u\alpha + \varphi) = 0.15 \text{ from which } u\alpha = 7.976 \qquad (4)$$

From (1) $-e^{-\alpha(0.8 - u)} = \ln\ 0.999$

$$-\alpha(0.8 - u) = \ln(-\ln 0.999)$$

$$\alpha(0.8 - u) = 6.907 \qquad (5)$$

Substituting (4) in (5) and solving we get $u = 0.43$ and $\alpha = 18.60$

Therefore the 1 year distribution is given by

$$F_{Yf}(y) = \exp[-e^{-18.60(y - 0.43)}]$$

For 50 years lifetime, the distribution is given by:

$$F_{Yf}(y) = \exp[-e^{-18.60(y - 0.43)}]^{50}$$

On simplifying we get $F_Y(y) = \exp[-e^{-18.60(y - 0.64)}]$

Therefore, the mean live load of the type 1 distribution is 0.64 kPa.

(p) Yield Stress of Steel

Erasmus and Smail (1989) in their statistical study of BHP grade 250 and 350 structural steel reported that the yield stress of BHP grade 250 structural steel is normally distributed with a mean value of 290MPa and standard deviation of 13MPa. The characteristic yield stress is 269MPa. As less than 5% of the steel section is expected to have a yield stress less than 269MPa, the normal distribution is truncated at a minimum value of 225MPa and maximum value of 340Mpa to ensure that realistic values are used in the simulation.

(q) **Time to Ceiling Failure**

The nominal fire resistance rating for Gib-board is FRR 30/30/30 (Winstone1997). In the deterministic assessment, the ceiling and wall cladding material is conservatively assumed to fail at 20 minute after commencement of fire. For probabilistic risk assessment, a scenario analysis should be done whereby the time to failure takes on a range of value. Hence the time to failure is assumed to be a random variable which exhibits normal distribution. The mean value is taken to be 20 minute and the coefficient of variation assumed to be 0.25. To be realistic, the minimum time to failure is fixed at 15 minute and the maximum time fixed at 30 minute. A truncated normal distribution is input into the spreadsheet.

(r) U Beam Section Modulus

The plastic section modulus is a geometrical property and is not expected to vary very much. However, data supplied by different manufacturers show slightly different value for the same nominal section. The plastic section modulus for 310 UB 40.4 manufactured by BHP is 633 x 10^3 mm^3 (BHP 1998). This is taken to be the mean value of a normal distribution for the section modulus with a coefficient of variation of 0.01. The distribution is truncated with a minimum value of 610 amd a maximum value of 655 x 10^3 mm^3.

(s) Thermal Conductivity of Ceiling Material

The ceiling is made from Gib board with thermal conductivity assumed to be 0.20 W/mK (Winstone 1997). This value is also assumed to be the mean value of the random variable - thermal conductivity of ceiling material. The distribution is assumed to be normal with a coefficient of variation of 0.05. The minimum value is assumed to be 0.16W/mK and the maximum to be 0.24W/mK.

(t) Thickness of Ceiling Material

The nominal thickness of the Gib-board is 0.0125m. This is taken as the mean value of an assumed normal distribution for the thickness. As the thickness is not expected to vary much, the coefficient of variation is assumed to be 0.05. The minimum value is taken to be 0.0106m and the maximum to be 0.0144m.

(u) Thermal Conductivity of Sprayed on Protection Material

The selected protection material is a sprayed on mineral fibre. The thermal conductivity is assumed to be 0.10W/mK (Buchanan 1999). This is also taken to be the mean value of the assumed normal distribution for this random variable.

Because of the greater variation in material properties, the coefficient of variation is assumed to be 0.15.

(v) Specific Heat Capacity of Sprayed on Protection Material

The specific heat capacity of the protecting material is assumed to be a normally distributed random variable with a mean value of 1100J/kgK (Buchanan 1999). The coefficient of variation is assumed to be 0.15.

(w) Thickness of Sprayed on Protection Material

This is assumed to be a random variable with value assumed to range from 0.006m to 0.015m. The mean value of the distribution is assumed to be 0.010m. As the thickness of insulation is dependent on the skill of the spray painter and the consistency of the material, this random variable is expected to show considerable variation. The coefficient of variation is thus assumed to be 0.20.

(x) Density of protection material

There are different types and brand of spray mineral fiber material suitable for protecting steel. To account for the variation, the density of material is assumed to be a normally distributed random variable. For this modelling the mean density of the protection material is taken to be 300 kg/m^3 (Buchanan 1999). A wide variation in the value of the density for different brands is expected. Therefore the coefficient of variation for this distribution is assumed to be 0.20.

The random variables for the complete working model have now being defined. No change is required for any of the formula calculations in the spreadsheet. Monte Carlo simulation can now take place.

9.3 RUNNING THE SIMULATION

With the @RISK model set up it is necessary to carry out the following before running the Monte Carlo simulation:

- Select the outputs of interest from the simulations. In this case, the outputs of interest are those defined in equations (9.1) and (9.2), that is, for each steel element, the maximum steel temperature, limiting steel temperature,

temperature safety margin, the fire resistance rating, time equivalence for fire severity and the time safety margin.

- Select the simulation settings which include: fixing the random number seed generator so that the whole series of simulation can be repeated under the same conditions; fixing the number of iterations to achieve desired accuracy; select sampling technique - Monte Carlo or the more modern Latin Hypercube sampling.

9.3.1 Analysing Simulation Outputs

The @RISK software presents its simulation outputs in the form of complete statistical data on all the basic (input) variables and the selected outputs. Properties like the mean, standard deviation, skewness, kurtosis, percentile values and so on are displayed. A sample output is shown in Appendix II. Other important functions of the @RISK program are generation of frequency histograms of the outputs, sensitivity analysis, scenario analysis, and others. The output function of interest in this project is the frequency histogram of any output (and input) which can be formatted into line graph to obtain the relative frequency distributions or the cumulative distribution function. As an example the relative frequency distributions of the maximum steel temperature and the limiting steel temperature for a 310 UB 40.4 beam protected by spray on mineral fibre are shown in fig.9.3.

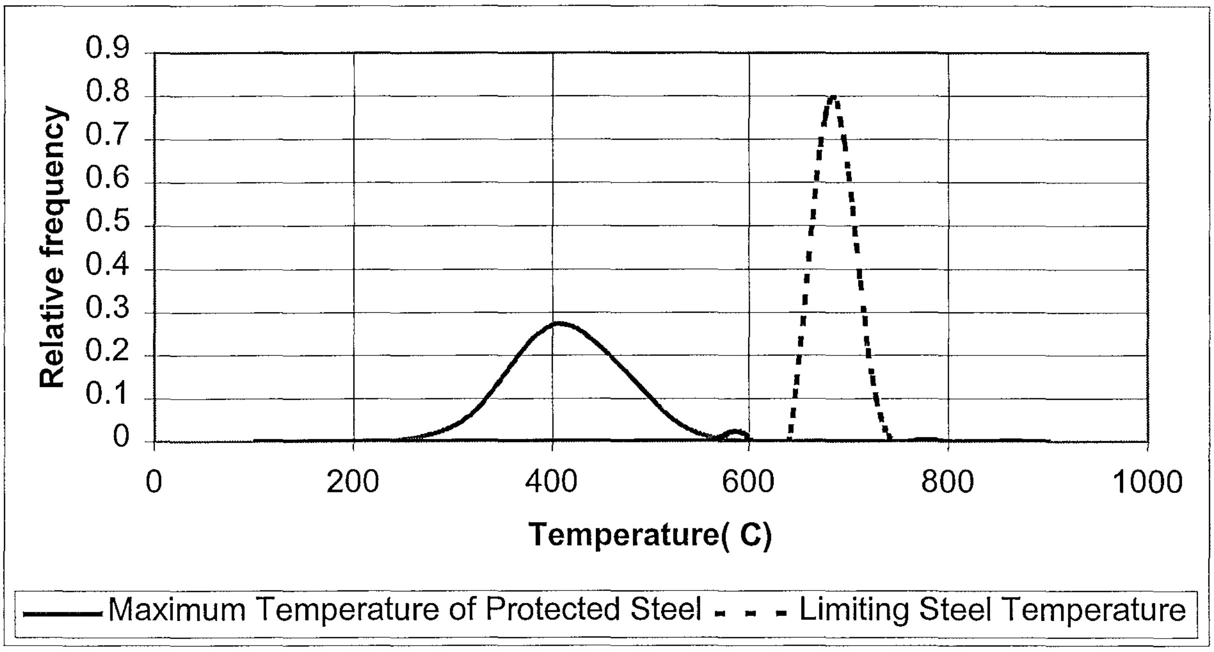


Figure 9-3Relative Frequency Distribution of Maximum and Limiting Steel Temperature for a Protected Beam EF (310 UB 40.4)

Fig.9.3 clearly depicts the reliability problem here as a capacity-demand problem. However, reliability cannot be directly deduced from Fig.9.3. It can be more easily deduced from the cumulative distribution function of the temperature safety margin.

9.3.2 Reliability Analysis Results

9.3.2.1 Probability of Failure

Several simulations were carried out for different structural steel members in the selected fire compartment. The output of interest here is the cumulative distribution (CDF) of the temperature safety margin and the resistance safety margin. The probability of failure of these structural members can be deduced from the CDF of the members. A typical CDF for the case of the beam EF (310 UB 40.4) protected only by the Gib-Board ceiling is shown in Fig.9-4.

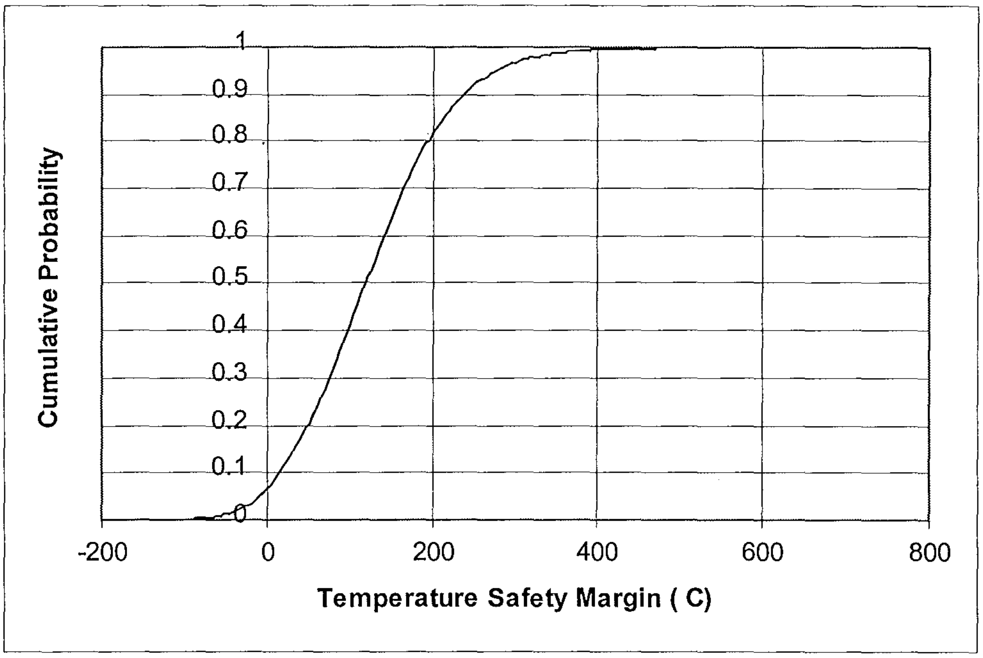


Figure 9-4 Cumulative Distribution Function of the Temperature Safety Margin for Unprotected but Ceiling Shielded Beam EF (310 UB 40.4)

From Fig.9-4 the probability of failure is worked out to be 0.0673, giving a β value of 1.5. Simulations and analysis in both the temperature and time domains were performed for all structural elements in the fire compartment for the case of unprotected, shielded and protected members and the results are as follows.

9.3.2.2 Temperature Domain Analysis

Under the temperature domain analysis, failure occurs when the temperature safety margin is less than or equal to zero, that is:

from equation (9.1) $$T_M = T_l - T_{max} \leq 0$$

Where: T_M = Temperature Safety Margin

T_l = Steel limiting temperature

T_{max} = Maximum steel temperature

The probability of failure given the outbreak of a range of fully developed fires is the probability of the temperature safety margin being less than or equal to zero, that is:

$$\text{Probability of failure, } P_f = P(T_l - T_{max} \leq 0)$$

The probability of failure for various steel members with different protection systems and when exposed to a range of parametric fires is shown in Table 9-1.

Table 9-1 Probability of Failure (P_f) and Reliability Index (β) of Members with Different Protection Systems -Analysis in Temperature Domain

	Unprotected		Ceiling/wall Shielded		Protected	
Steel Member	P_f	β	P_f	β	P_f	β
Beam AB	1.41×10^{-3}	2.98	2.69×10^{-4}	3.42	0	6
Beam AC	7.72×10^{-4}	3.18	1.76×10^{-4}	3.57	0	6
Beam BD	5.11×10^{-5}	3.88	1.34×10^{-5}	4.20	0	6
Beam EF	0.31527	0.48	0.0728	1.46	0	6
Column AA'	2.79×10^{-5}	4.05	0	6	0	6
Column BB'	0	6	0	6	0	6
Column CC'	3.62×10^{-2}	1.79	2.59×10^{-3}	2.80	0	6
Column DD'	6.84×10^{-5}	3.81	2.64×10^{-5}	4.05	0	6

Table 9-1 shows that the two members with the least fire resistance are the beam EF and column CC'. Beam EF has around 32% probability of failing if it is unprotected and around 7% probability of failure if shielded by the Gib-board ceiling. Column CC' has 4% probability of failure if unprotected and 0.3% of failure if shielded by Gib-board wall claddings. Analysis in the temperature domain also shows that the probability of failure can be very low even for unprotected members. Whether this will result in unsafe design would be discussed later. For all cases of protection by sprayed on mineral fibre of around 10mm thickness, the probability of failure has gone down to practically zero. Reliability index value of 6 is fixed for cases where the probability of failure is practically zero.

9.3.2.3 Time Domain Analysis

In analysis under the time domain, the failure criterion is:

$$t_l - t_e \leq 0$$

where: t_l is the time to reach limiting temperature (fire resistance)

t_e is the time equivalence (fire severity)

As explained in section 8.3.9, t_l the "time to reach limiting temperature" can be obtained by two methods:

(a) Empirical correlations (SNZ 1994):

For 3 sided exposure, from equation (8.25)

$t_l = -5.2 + 0.0221T_l + 0.0\text{-}3.40\ T_l\ V/F$

For four sided exposure, from equation (8.26)

$t_l = -4.7 + 0.0263T_l\ + 1.67T_l\ V/F$

For ease of reference, t_l obtained from these correlations are designated as t_l(formula)

(b) The value of t_l can also be obtained from the spreadsheet for the fire model, heat transfer model and the structural response model explained in section 8. For ease of reference, t_l obtained from spreadsheet is designated as t_l (cal)

There are also two methods for calculating the time equivalence t_e, viz from the spreadsheet, designated here as t_e(cal), and by using the Eurocode formula (equation 4.6), designated here as t_e (EC1)

Therefore, with two methods each of obtaining t_l and t_e there are four combinations of the limit state equation. They are designated here as:

Time Domain 1: t_l (formula) – t_e(cal)

Time Domain 2: t_l (cal) – t_e(cal)

Time Domain 3: t_l (formula) – t_e(EC1)

Time Domain 4: t_l (cal) – t_e(EC1)

The probability of failure of a steel member, analysed under any of the time domain designated from 1 to 4 is the probability of violation of the limit state equations.

The results for analyses under the time domain for the steel member (beam EF) with the least fire resistance are presented in Table 9-2. For comparison, the result from temperature domain analysis is included in the Table.

Table 9-2 Probability of Failure (P_f) and Reliability Index (β) of Beam EF with Different Protection Systems – Analysis in the Time Domain

	Unprotected		**Ceiling Shielded**		**Protected**	
Failure Criteria	P_f	β	P_f	β	P_f	β
Time Domain 1 t_l (formula) - t_e (cal) ≤ 0	0.9747	0	—	—	0.0020	2.88
Time Domain 2 t_l (cal) - t_e (cal) ≤ 0	0.9986	0	0.8294	0	0	6
Time Domain 3 t_l (formula)- t_e (EC1) ≤ 0	0.5853	0	—	—	0.0237	1.98
Time Domain 4 t_l (cal) - t_e (EC1) ≤ 0	0.8169	0	0.4211	0.20	0.0043	2.63
Temperature Domain $T_l - T_{max} \leq 0$	0.31527	0.48	0.0728	0.48	0	6

The followings are observed from the data in Table 9-2:

- Compared with the temperature domain analysis, the probability of failure has increased substantially when analysis is carried out in the time domain for all failure criteria.

- In particular, the analysis shows that the unprotected beam EF is almost certain to fail when the time equivalence (t_e (cal)) is computed from the spreadsheet.
- The probability of failure is decreased when the time equivalence (t_e(EC1)) is calculated from the Eurocode formula.
- The formula for determining the time to reach limiting temperature agrees well with the spreadsheet method for calculating the same.

The differences in results between the two domains and within the time domain (for different criteria) would become more apparent when one looks at the results for all structural members in the fire compartment.

9.3.2.4 Comparison of Results from Different Domains of Analysis

The probability of failure of all steel members, analysed under the two domains, are plotted as bar charts in order to better see the differences between them. Figure 9-5 shows the bar chart for the case of unprotected steel beams.

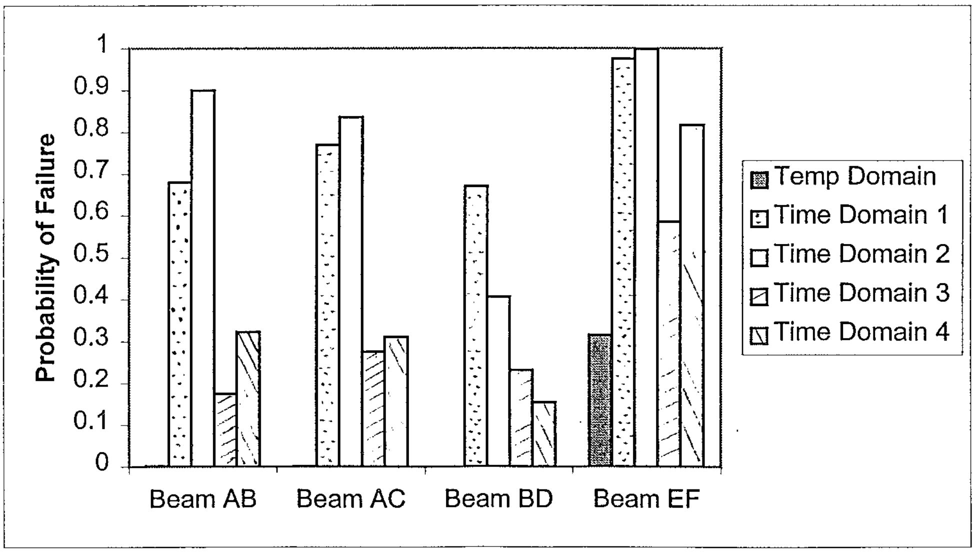


Figure 9-5 Probability of Failure of Unprotected Steel Beam Exposed to Parametric Fire - Results from Different Analysis Domains

The bar chart for the case of unprotected columns is shown in Figure 9-6.

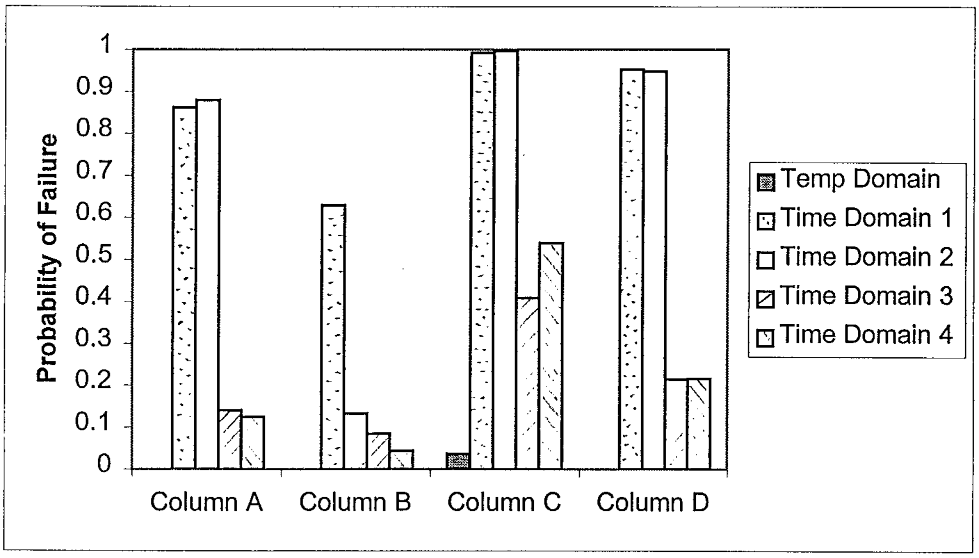


Figure 9-6 Probability of Failure of Unprotected Steel Columns Exposed to Parametric Fires - Results from Analysis under Different Domains

Examination of figure 9-5 and 9-6 reveal the following:

- Analysis in the temperature domain shows the lowest probability of failure for all members. Except for beam EF and column CC', the probability of failure for all unprotected members is practically zero. This means that temperature domain analysis gives the least conservative results.
- Analysis in the "time domain 2" (where both the t_l and t_e are calculated by spreadsheets) results in the highest probability of failure for all members. This method of analysis thus gives the most conservative results.
- The empirical correlations provided by NZS 3404 (SNZ 1997) for time to reach limiting temperature ("time domain 1") agree very well with the spreadsheet method of calculation ("time domain 2").

- The usage of the Eurocode formula (EC1 1994) for time equivalence has resulted in the reduction in the probability of failure, as shown by the results for "time domain 3" and "time domain 4" analyses. This trend confirms the same unconservative results from the deterministic calculations in section 8.5.2.

9.3.2.5 Results for Protected Steel Members

(i) Unprotected but Shielded Members

The result for the case where the steel members are unprotected but shielded from the fires by suspended Gib-board ceilings or wall claddings are shown in figure 9-7 and 9-8. Figure 9-7 shows the results for steel beams shielded by suspended Gib-board ceiling which is assumed to collapse 20 minutes after commencement of fire.

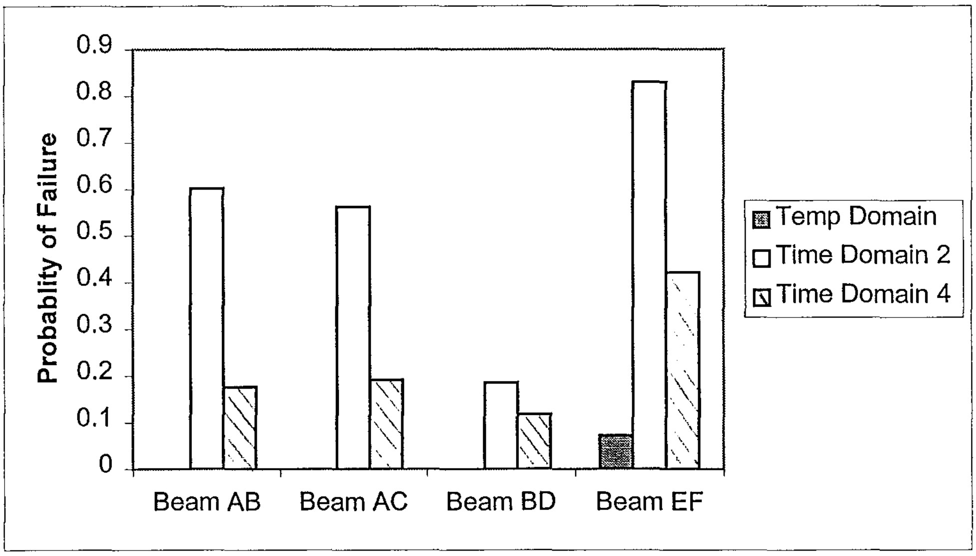


Figure 9-7 Probability of failure of "Shielded" Steel Beams Exposed to Parametric Fire - Results from Different Analysis Domains

"Time domain 1 and 3 analysis" are not applicable in this case. The results show that using a suspended Gib-board ceiling with a fire resistance of around 20 minutes to shield the beams from direct heating of the fire has not appreciably improved the performance of the beams under fire condition. This is not susprising given that the time equivalent ranges from 29 minutes (Eurocode formula) to 36 minutes (spreadsheet calculation for protected beam AB) in the deterministic calculations. The results from the Monte Carlo simulation is expected to be worse because of the wider range of values of parameters used in the iterative calculations.

The case of steel columns shielded by Gib-board wall claddings from direct heating by the fire is shown in .figure 9-8.

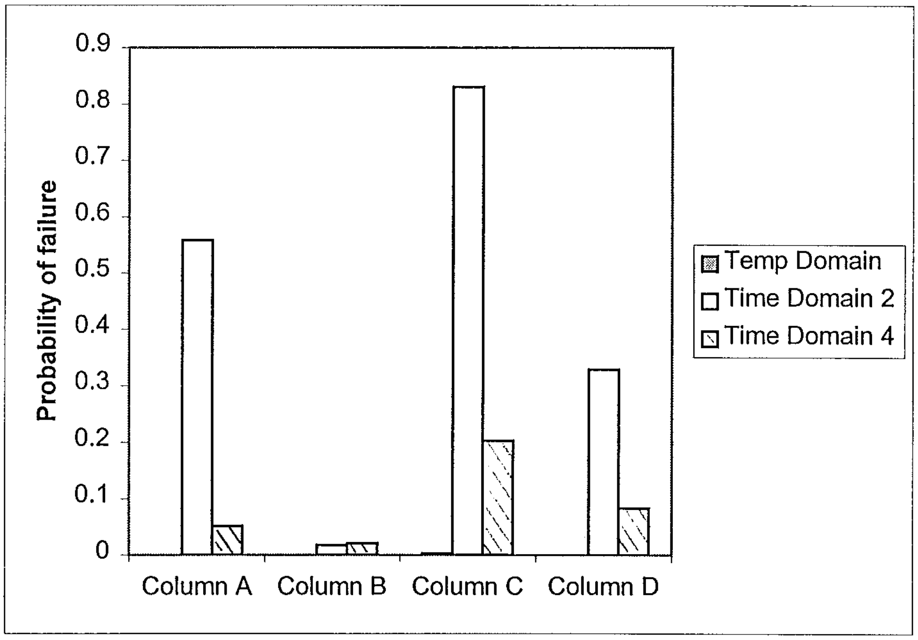


Figure 9-8 Probability of Failure of "Shielded" Columns Exposed to Parametric Fires - Results from Different Analysis Domains

Figure 9-8 again shows that there is no substantial improvement in the column performance if the wall claddings only have a fire resistance of around 20 minutes. More will be discussed on the role of suspended ceiling and wall claddings as passive protection systems in the general discussion in the next section.

(ii) Protected Steel Members

Except for the case of protected beam EF whereby the probability of failure of failure ranges from 0.2% to around 2.4% when analysed under the time domain, the probability of failure for all other protected steel members is practically nil, irrespective of the methods of analysis.

9.4 DISCUSSION

Analysis of structural reliability of individual steel members in a representative hotel room when exposed to a range of possible parametric fires has been carried out. A qualitative comparison with the results of the deterministic calculations in chapter 8 show that more members can be considered unsafe when reliability analysis has been carried out. However, a definite conclusion cannot yet be made until the acceptable probability of failure and safety index has been fixed. This will be done in the next chapter. At the same time the results of this reliability analysis are far from being absolute and should only be treated as being nominal or indicative for the following reasons:

- All calculated probabilities of failure (and therefore the reliability index) are conditional probabilities given the outbreak of a fully developed fire. The calculation of the overall probability of failure would incorporate factors like probability of ignition, flashover, operation of active protection system, response of the fire services and other factors. This will be discussed further in the next chapter.
- The structural response and hence the reliability of the whole structure or even sub-assemblies are very different from that of individual members. In fact, the sub-assemblies or the whole structure has a greater reserve of strength under elevated temperature compared to that estimated for individual members (Clifton 1998). This is due mainly to the structural continuity and redundancy found in the whole structure which allows for load sharing effect under any loading condition, including thermal loading (Buchanan 1999).

- Notwithstanding the above paragraph, analysis of global behaviour of whole structures goes well beyond the requirement of most current regulatory approval systems which look only at isolated members and not on their interaction (IEA 1989). Computer programs like FASBUS (AISI 1980) and CEFICOSS (Shleich 1987) are available to evaluate structural adequacy of an entire building with fire in one compartment. However, Sullivan et al (1994), in their review of 13 such programs, concluded that none of them are user friendly or sufficiently well documented for routine use as design tools.
- The methodology for structural fire design elaborated in this report constitutes only a small part of the overall design strategy as outlined in chapter 2. The overall design strategy requires a wholistic approach that takes into account factors like overall layout, compartmentation, performance of non-structural building elements, active and passive protection systems, human behaviour and a host of other factors. And, mainly for economic reason, a trade off can be struck between the various protective features. The latter aspect certainly requires experience and ingenuity.
- Because of the greater uncertainties associated with fire science and engineering at this point in time, it is safer to overdesign. The fact that whole unprotected steel structure has greater fire resistance than the individual members (as proven in tests or actual cases) may not be enough justification to forego, as a cost saving measure, passive protection system.
- The use of suspended ceilings in structural fire protection has become an accepted practice in many countries (Read & Adams 1979, Fitzgerald 1997, Feeney 1998). Pettersson, et al (1976) worked out a method of calculating heat transfer through the ceiling but the technique is considered too complex for simple design work. The assumption that steel beams are lightly protected for only the first 20 minutes in the original design and in this project may be too conservative. If the ceiling is to be depended upon as a protective barrier, the most important thing is to ensure that the suspended ceiling is designed and constructed to be fire rated (Fitzgerald 1997).
- **Because of the limitations described in Chapter 8, only the temperature domain results in this chapter should be used for assessing actual designs.**

10 TARGET RELIABILITY

10.1 PRELIMINARY

Before attempting to relate the results of the two case studies undertaken in this project to issues like risk acceptability and target reliability, it is deemed necessary to address fundamental questions about the meaning of related terms like "failure" and "probability of failure".

10.1.1 Meaning of Failure

What is failure? In engineering practice, failure is what the engineer defines it to be and nothing else. For example, if the stress induced by an earthquake exceeds the ultimate stress of the material, it could be called a failure. Alternatively, if the stress exceeds the yield stress of the material it could also be called failure. In both cases it is not necessary for the building to collapse like a pack of cards for the meaning of failure to be appreciated. Similarly, in structural steel fire design, if the failure criterion of the maximum steel temperature exceeding the limiting steel temperature is met, the member should be deemed to have failed absolutely without the need to physically check for signs of crack, excessive deflection and so on. The same reasoning applies when the failure criterion is the time equivalence exceeding the fire resistance rating. This qualification of the meaning of failure is necessary to reduce any doubt or argument as to whether failure can or has occurred. The underlying analyses can be questioned or the criterion changed; but once the criterion is fixed or agreed upon, it should become the basis for either design acceptance or reassessment/ redesign.

10.1.2 Probability of Failure

What does the calculated probability of failure P_f mean? Can it be related to observed rates of failure for actual structures under fire situation? How can a knowledge of P_f helps in achieving safer or more economical structures? These are important questions and ones about which a degree of controversy and disagreement still exists (Melchers 1987), and will not be discussed here. Suffice to say that there is no such thing as absolute probability of failure simply because it is impossible to account for all uncertainties. Therefore, reliability analysis considering only a subset of uncertainties will result in a probability estimate which should be termed a "nominal" measure (Melchers 1987). This term has been applied to results of the two case studies in this project.

10.1.3 Acceptable Probability of Failure

When a reliability assessment has been performed it must be decided whether the calculated probability of failure is acceptable. Acceptable probability of failure is seldom encoded for a specific engineering interest, for example, structural collapse during fire. A common criterion for risk acceptability is to compare the calculated probability of failure with societal risk (Rowe 1977).

An attempt will be made in the next section to calculate the acceptable probability of failure for accidental building fire situation and the results of the reliability analyses in the two case studies will be assessed.

10.2 CALCULATION OF ACCEPTABLE PROBABILITY OF FAILURE

In the two case studies (chapter 7 and 9) the probability of failure of building elements under fire condition have been calculated. The next important question is what constitutes acceptable probability of failure and reliability index?

A document that provides a methodology for calculating the acceptable probability of failure is the *Design Guide - Structural Fire Safety* published by the Conseil International du Bâtiment (CIB 1986). The following are adapted from Appendix 5: *Safety Factors γ and Differentiation Factors γ_n* of the document.

10.2.1 Failure Probabilities

The probability for attaining a limit state defined, for example, by the failure condition M = R - S $\leq$ 0 is calculated by considering the probability distribution functions of the random variables R and S. The probability of failure P_f is expressed as:

$$P_f = P(R - S \leq 0) \qquad (10.1)$$

Methods of calculating the probabilities are the first-order second moment method (FOSM) and the method of Monte Carlo simulation as described in chapter 6. If $P_{f,a}$ and β_a represent the acceptable failure probability and safety index respectively, the reliability verification is accomplished by ensuring that:

$$P_f \leq P_{f,a} \text{ or } \beta \geq \beta_a \qquad (10.2)$$

10.2.2 Acceptable Failure Probabilities

The failure probability decisive for the (accidental) fire design situation may be specified as :

$$P_{f,a} = \frac{P_f}{P_a} f(A) \qquad (10.3)$$

Where P_f is the acceptable failure probability for a certain reference period. Different value may be allocated to different safety classes - depending on the consequence of failure. Example values for acceptable lifetime (50 years) failure probabilities are listed in Table 10-1.

Table 10-1 Acceptable lifetime failure probabilities* (CEB 1976)

Expected number of fatalities in case of fire	Economic losses		
	Low	Medium	High
Low (≈ 0.1)	10^{-3}	10^{-4}	10^{-5}
Medium (≈ 1.0)	10^{-4}	10^{-5}	10^{-6}
High (≈ 10.0)	10^{-5}	10^{-6}	10^{-7}

* All values are indicative and have to be checked on a national basis

P_a is the probability of occurrence of severe fires within the reference period considered. A simple estimate is obtained from

$$P_a = P(\text{fire})P_1P_2\ldots. \qquad (10.4)$$

with P(fire) representing the probability of (initial) fire outbreak, which depends on the occupancy and size of the compartment. In a general presentation P(fire) can be modelled by the following (Rutstein & Clarke1979):

$$P(\text{fire}) = pA^X \qquad (10.5)$$

Where p denotes the probability of occurrence per m^2/year.

A is the total fire compartments area (m^2)

X is an index with value ≤ 1.0

Assuming X = 1.0 would therefore be conservative. Example values for p (per m^2 and year) are given in Table 10.2.

Table 10-2 Probabilities of Occurrence of Fires* (Rutstein & Clarke 1979)

Occupancy	Probability per m^2 and year
Dwelling	10^{-5}
Offices	10^{-6}
Industrial buildings	10^{-6}

*All values are indicative only and have to be checked on national basis.

P_1, P_2 .....identify the decrease of probability for an initial fire to develop into a severe fire, depending on the various fire detection and fire fighting provisions employed. Indicative values are shown in Table 10.3.

Table 10-3 Reduction of Probability for a Severe Fire* (Bub et al 1979)

Fire Safety Provision	P_i
Average standard public fire brigade	0.1
Adequately maintained sprinkler system	0.02
Factory/private fire brigade, depending on standard	0.5 - 0.05
Adequately maintained detection and alarm system	1 -0.1

*All values are indicative and have to be checked on a national basis

However, if several safety provisions are employed, the product P_1P_2.... should be associated with a lower bound to account for the dependency among the provisions with regards to their possible success.

A function f(A) may be introduced, if desired, to account for a risk being enhanced as compartment size is increased, e.g. as

$$f(A) = A^*/A \tag{10.6}$$

with A* corresponding, for example, to the average compartment size for a certain type of occupancy.

As an illustration, the acceptable probability of failure for the building in Case Study II will be calculated based on this methodology.

10.3 APPLICATION TO CASE STUDY II

10.3.1 Calculation of Acceptable Probability of Failure

(a) Relevant Building Information

- Purpose Group SA - sleeping in accommodation facility (BIA 1995)
- Estimated total compartment floor area (4 storeys extension only): $3600 m^2$

(b) Tolerable Lifetime Failure Probabilities P_f

- Economic losses - considered high for hotel business
- Expected number of fatalities - medium to high

Two cases will be considered: (i) medium fatalities/high economic losses and (ii) high casualties/high economic losses.

For case (i) the approximate annual acceptable probability of failure P_f is obtained from Table 10.1 by dividing the lifetime value by 50 to give a value of 2×10^{-8}

For case (ii) the value of $P_f = 2 \times 10^{-9}$.

(c) Probability of Occurrence of Severe Fire P_a

From Table 10.2 the probability of occurrence of fire per m^2 per year, p, is taken as 10^{-5} (dwelling).

The probability of initial fire P(fire) = PA^X

$$= 10^{-5}(3600)^1$$

$$= 0.036.$$

From Table 10.3 the following reduction of probability are assumed:

- Average standard public fire brigade $\Rightarrow P_1 = 0.1$
- Adequately maintained sprinkler system $\Rightarrow P_2 = 0.02$
- Adequately maintained detection and alarm system $\Rightarrow P_3 = 0.01$

$\therefore$ the annual probability of occurrence of severe fire $P_a = P(\text{fire})P_1P_2P_s$

$$= (0.036)(0.1)(0.02)(0.1)$$
$$= 7.2 \times 10^{-6}$$

(d) Value of Function f(A)

As data on "average fire compartment size" is not available, the function f(A) will be dispensed with.

(e) Acceptable Probability of Failure and Reliability Index

Substituting the values of P_f and P_a calculated above into equation (10.3), the following acceptable probability of failure are obtained:

Case (i) Medium fatalities/high economic losses: $P_{f,a} = 2.77 \times 10^{-3}$

The corresponding acceptable reliability index $\beta_a = 2.77$

Case (ii) High fatalities/high economic losses: $P_{f,a} = 2.77 \times 10^{-4}$

The corresponding acceptable reliability index $\beta_a = 3.45$

10.3.2 Qualifying Comment

It should be stated that the failure stated in Table 10.1 is not structural failure under fire condition. It is more of a societal nature with human fatalities and economic losses as the principal determinants. However, due to the dearth of statistical data on structural failure in fire, the probability of failure as expressed in equation (10.3) would adopted for the present analysis.

10.3.3 Target Probability of Failure and Reliability Index

The target probability of failure and reliability index for any situation can only be set after sufficient number of similar analyses as done in section 10.3.1 have been carried out to yield a large enough database. For illustration purpose, the average of the probability of failure from case (i) and (ii) in section 10.3.1 will be taken as the target values:

Target Probability of failure $P_{f,t} = 1.52 \times 10^{-3}$

Target Reliability Index $\beta_t = 2.96$

10.4 RELIABILITY ASSESSMENT OF CASE STUDY I

In Case Study I the reliability index of the performance of the concrete slab was calculated to be 2.89. Assuming that the fire compartments' parameters, fire behaviour and risk criteria in Case Study II are applicable, this value of β falls short of the target reliability index of 2.96. The concrete is thus considered unsafe in its present design.

10.5 RELIABILITY ASSESSMENT OF CASE STUDY II

With the target probability of failure and reliability index thus fixed the performance of the steel members in Case Study II would be re-examined. The results of assessment of the temperature domain calculation is shown in Table 10-4.

Table 10-4 Members' Performance (Temperature Domain Analysis) Assessed Against Target Reliability Index

	Unprotected		**Ceiling/wall Shielded**		**Protected**	
Steel Member	β	**Remark**	β	**Remark**	β	**Remark**
Beam AB	2.98	safe	3.42	safe	6	safe
Beam AC	3.18	safe	3.57	safe	6	safe
Beam BD	3.88	safe	4.20	safe	6	safe
Beam EF	0.48	unsafe	1.46	unsafe	6	safe
Column AA'	4.05	safe	6	safe	6	safe
Column BB'	6	safe	6	safe	6	safe
Column CC'	1.79	unsafe	2.80	unsafe	6	safe
Column DD'	3.81	safe	4.05	safe	6	safe

Because a range of performance is involved, the term "safe/unsafe" rather than fail/ok is used. From Table 10-4 it can now be ascertained that steel member EF performance is unsafe under fire condition. This beam is one of the intermediate beams that support the concrete floor slab.

The same target reliability index will be applied to analyses done under the time domain in Case Study II. In this assessment, the most conservative and the least conservative time domain analyses will be looked into. The most conservative time domain analysis refers to the case where both the "time to attain limiting temperature" and the time equivalence are calculated from the spreadsheet. The least conservative case is where the "time to attain limiting temperature" is calculated from the spreadsheet and the time equivalence is obtained from the Eurocode 1 formula (EC1 1994).

The reliability assessment of the "most conservative" time domain analysis is shown in Table 10-5.

Table 10-5 Members' Performance (Most Conservative Time Domain Analysis) Assessed against Target Reliability Index

	Unprotected		**Ceiling/wall Shielded**		**Protected**	
Steel Member	β	**Remark**	β	**Remark**	β	**Remark**
Beam AB	0	unsafe	0	unsafe	6	safe
Beam AC	0	unsafe	0	unsafe	6	safe
Beam BD	0.24	unsafe	0.89	unsafe	6	safe
Beam EF	0	unsafe	0	unsafe	6	safe
Column AA'	0	unsafe	0	unsafe	6	safe
Column BB'	1.12	unsafe	2.11	unsafe	6	safe
Column CC'	0	unsafe	0	unsafe	6	safe
Column DD'	0	unsafe	0.45	unsafe	6	safe

Table 10-5 shows that only steel members protected by sprayed on mineral fiber can be considered safe. All unprotected members, even if they are shielded by suspended Gib-board ceiling or Gib-board wall linings, are deemed to be unsafe.

The same assessment is applied to the case of the "least conservative" time domain analysis and the results shown in Table 10-6

Table 10-6 Members' Performance (Least Conservative Time Domain Analysis) Assessed against Target Reliability Index

	Unprotected		Ceiling/wall Shielded		Protected	
Steel Member	β	**Remark**	β	**Remark**	β	**Remark**
Beam AB	0.41	unsafe	0.93	unsafe	6	safe
Beam AC	0.52	unsafe	0.87	unsafe	6	safe
Beam BD	1.02	unsafe	1.19	unsafe	6	safe
Beam EF	0	unsafe	0.20	unsafe	6	safe
Column AA'	1.15	unsafe	1.63	unsafe	6	safe
Column BB'	1.71	unsafe	2.04	unsafe	6	safe
Column CC'	0	unsafe	0.83	unsafe	6	safe
Column DD'	0.82	unsafe	1.38	unsafe	6	safe

The results from Table 10-6 are not significantly different from the results shown in Table 10-5.

10.6 DISCUSSION

In this project the structural performance of steel members under different fire protection regimes and exposed to "real" fire was first analysed deterministically. Under the temperature domain analysis, the results showed that only one of the unprotected members would fail under fire condition but all unprotected members are safe if the shielding effect of the ceiling and wall linings were accounted for. The results for time domain analyses showed a substantial increase in the number of members that would fail under fire condition, even when the shielding effect of the ceiling and wall linings were taken into account. In both domains of analysis for members protected by sprayed on mineral fiber there was no case of failure.

Reliability analysis using Monte Carlo simulation was then carried out on the deterministic calculations. The nominal probabilities of failure of the steel members under different protection regimes and exposed to a range of "real" fires were calculated. The overall results show that a substantial number of members deemed to be safe under the deterministic assessment have relatively high probabilities of failure. The final reliability assessment could only be done when the acceptable probabilities of failure have been established.

The acceptable probabilities of failure and reliability index have been worked out in this chapter and reliability assessment carried out in section 10.4 and 10.5. Considering only results from the temperature domain analysis, the reliability assessment in Table 10.4 shows that:

1. Even if all members are totally unprotected, several have a high enough safety index to be considered safe, but some members are very unsafe.

2. Considering ceiling or wall protection, the safety indices increase, but two members are still marginally unsafe.

3. If all members are protected with spray-on fire protection, the resulting safety index is very high.

11 CONCLUSIONS

This project looks into three main areas viz (1) fire engineering design of a steel framed building, (2) reliability analysis of the design and (3) reliability assessment of results from (2). The conclusion from each area and the general conclusion are appended below:

11.1 CONCLUSION FROM FIRE ENGINEERING DESIGN

This report has shown how it is possible to carry out fire engineering calculations to assess whether a simple structural steel member has sufficient strength to carry loads throughout a fire. The steel member may be unprotected, protected by wall and ceiling lining materials, or protected by an applied proprietary product.

11.2 CONCLUSION FROM RELIABILITY ANALYSIS OF DESIGN

This report has shown how a deterministic assessment of steel member performance can be converted into a reliability assessment, taking into account the inherent variability of all of the factors considered in the design.

This report has made calculations in both the temperature domain and the time domain, but only those in the temperature domain are considered sufficiently accurate for conclusions to be drawn.

11.3 CONCLUSION FROM RELIABILITY ASSESSMENT

In this project an indicative target probability of failure and reliability index have been worked out. Based on this target reliability index the conclusion that can be drawn is that some unprotected steel members will be unsafe, most members protected with wall or ceiling linings will be safe enough and all members protected with applied fire protection will be very safe.

This conclusion is hypothetical, as it is dependent on a large number of assumptions as described in this report.

11.4 GENERAL CONCLUSION

Deterministic or "single value" analysis in design can give very misleading results if the variability in properties and the full range of possible scenarios are not taken into account. This is especially true for fire engineering design for the very reason that fire behaviour and its effect on material properties have a lot of scatter in the data. Under such circumstances, the application of reliability assessment would highlight any deficiency or shortcoming in the design and provide a good basis for re-examining of the design to meet the performance requirements

This report has shown how a simple deterministic calculation can easily be extended into a reliability analysis using readily available computer software packages.

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